PRELIMINARY STORMWATER MANAGEMENT DESIGN REPORT

ANJUMAN E BURHANI

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1/27/17

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May 22,2013
August 6, 2013
November 1, 2013
March 31, 2016
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January 27, 2017

This report has been prepared by Staff of DCI Engineers under the direction of the professional engineer whose stamp and signature appears hereon.

Contents

PROJECT OVERVIEW

The proposed Anjuman E Burhani project will construct a mosque and community center with a 10,411 sq. ft. footprint on a 1.1-acre site in Redmond, Washington. The property has 32.5 ft. of frontage on NE 51st St. along its southern boundary and is bounded by a wedge of right-of-way on the remainder of its south side and by SR 520 on the west. Single family residences occupy parcels to the north and east. A triangular extension of a parcel to the north, 19.5 ft. at its widest, extends south 159 ft. along the project site's west boundary, separating it from the SR 520 right-of way.

The subject parcel slopes down to the north and west at approximately 6% and is substantially cleared. A derelict house occupies the center of the site, and a row of evergreen trees lines the driveway access from NE 51st Street. The remainder of the site is largely covered with dense grass and blackberry bushes.

A March 11, 2013 geotechnical report by Robinson Noble describes the predominant underlying soil type as glacial drift, consisting of dense silty sand and sandy silt. The report indicates that significant groundwater flows are not anticipated on this site.

The mosque will be constructed near the low end of the site, and the remainder of the property will be occupied by landscaping, access driveways and parking for 34 vehicles.

Stormwater will be collected by catch basins, routed through cartridge filter systems for water quality treatment, then held in an underground detention pipe and released at a metered rate as discussed in the "Flow Control Design" section below. The discharge from the detention facility will be pumped to be discharged north of the site to the City's system.

MINIMUM REQUIREMENTS

The mosque construction is classified as a Large Project because it will create more than 5000 sq. ft. of new impervious surface, and is required to meet all of the following minimum requirements as applicable.

1. Preparation of Stormwater Site Plans

Detailed stormwater site plans will be prepared in accordance with DOE and City of Redmond standards upon finalization of the site configuration.

2. Construction Stormwater Pollution Prevention

The site construction plans will include TESC provisions with notes and details. A Stormwater Pollution Prevention Plan (SWPPP) also will be prepared. The proposed BMPs will include siltation barriers, an armored construction entrance and inlet protection. Offsite streets will be monitored for tracking of mud and debris by construction vehicles.

3. Source Control of Pollution

The SWPPP will include provisions for materials handling and pollution source control during construction. Any hazardous material releases shall be contained, cleaned up, and reported. The SWPPP will provide details on how the following requirements will be met:

- Monitoring plan.
- Designated project contact.
- Secondary containment.
- Provisions to secure hazardous materials.
- Response to leaking vehicles and equipment.
- Practices and procedures regarding transfer of flammable and combustible liquids.
- On-site cleanup materials and other containment and cleanup provisions.

The operation of the mosque is not expected to generate significant pollutants other than parking lot runoff.

4. Preservation of Natural Drainage Systems and Outfalls

There are no surface channels on the project site. Runoff from the site appears to sheet flow across the properties to the north and into a stormwater system in a housing development on 154th Avenue NE. The developed site conditions will continue to discharge collected surface water runoff to the storm system in 154th Ave NE.

5. On-site Stormwater Management

The poor infiltration capacity of the native glacial till soil makes retention of stormwater on site impractical. The landscape areas will be enhanced with compost-amended soil, which will provide some runoff attenuation.

6. Runoff Treatment

Runoff from paved areas will be collected in catchbasins and routed through StormFilter cartridge treatment systems prior to detention.

7. Flow Control

All stormwater runoff that can feasibly be captured will be routed through an underground detention pipe and released at reduced rates. A detailed discussion of design considerations follows in the "Flow Control Design" section of this preliminary report.

8. Wetlands Protection

There are no wetlands on or near the project site.

9. Basin/Watershed Planning

The proposed project lies within the North Overlake Drainage Area and is therefore subject to the City of Redmond Alternative Flow Control Standard. A detailed discussion of design considerations follows in the "Flow Control Design" section of this preliminary report.

10. Operation and Maintenance

The mosque's maintenance staff will be charged with monitoring the function of the on-site drainage facilities. An Operations & Maintenance Manual will be prepared to provide guidelines for maintenance staff.

DOWNSTREAM FLOWPATH

Surface water runoff generated from the site sheet flows north. The surface water runoff drains to the storm drainage collection system located in 154th Ave NE. The storm system is part of a plat that was constructed in 1975. The storm drainage system is entirely piped.

The piped storm system drains north and stays within the roadway. An existing detention pond is located approximately 1,100 feet north of the site. The detention facility discharge pipe drains north and stays within 156th Ave NE right of way for an additional 300 feet (end of quarter mile downstream analysis). Ultimately, the piped system discharges to the WSDOT right of way.

CONVEYANCE DESIGN

The final drainage report will include conveyance sizing calculations, when the site configuration is finalized.

FLOW CONTROL DESIGN

The project site lies within Basin 4 of the North Overlake Drainage Area (see map in the appendix), draining to the WSDOT SR 520 storm trunk. Per City of Redmond standards, developed in consultation with WSDOT, development within this basin has the option of designing stormwater control to an alternative flow control standard rather than to the Department of Ecology's standards.

The alternative flow control standard provides a single-event peak flow rate criterion for stormwater released from the project site. In a 50-year recurrence storm, the site is permitted to release stormwater at a rate of 0.37 cfs per acre. There are no criteria for larger or smaller storms or for antecedent site conditions.

At full build-out, the project will create or replace approximately 0.82 acre (73% of the site) of impervious roof and pavement surface within the property boundary. Additionally, Redmond stormwater regulations require that runoff from half of NE 51st Street along the project's right-of-way frontage be managed as part of the project's stormwater basin.

Runoff from most paved areas will be collected in catch basins and routed to StormFilter cartridge facilities for water quality treatment before being conveyed to a detention pipe. Roof runoff will bypass the water quality treatment and will be routed directly to the detention pipe.

Because of the topography of the site and access limitations on the placement of the detention facility, runoff from approximately 25% of the on-site portion of the basin will bypass the detention tank. Also, all of the off-site portion of the basin in the NE 51st St. frontage will bypass the basin, continuing to drain directly to the ditch in the SR 520 right-of-way as it does currently. To account for this bypass, the 50-year peak flow rate was calculated separately for the bypass areas. This rate (0.24 cfs) was subtracted from the allowable full-basin release rate (0.46 cfs). The resultant rate (0.22 cfs) was used as the target release rate in sizing the detention pipe for the flow that reaches it.

The on-site areas that will not drain to the detention facility include the landscape area on the north side of the building and the driveways on the east and west sides of the building.

The detention pipe was sized with HydroCAD software to meet the 50-year single-event alternative flow control standard for release to the story system in 159th Ave NE and eventually the SR 520 right-of-way. Additional sizing analysis was performed to ensure the facility would not overflow in a 100-year recurrence storm.

The detention pipe will be 60 inches in diameter and 83 feet long, with a 0.5% slope and 6 inches of dead storage at the low end. Access manholes will be provided at both ends, accessible from a paved surface. The outlet control structure will include a single 2-inch orifice and a notch at the top of the riser.

The following table summarizes the output from the HydroCAD analysis:

TOTAL AREA: 1.24 AC	ALLOWABLE RELEASE RATE*	DETAINED SITE RUNOFF	BYPASS RUNOFF	COMBINED DESIGNED RELEASE RATE
50-YR Storm	**0.46 CFS**	**0.22 CFS**	**0.24 CFS**	**0.46 CFS**
2-YR Storm	N/A	0.13 CFS	0.10 CFS	0.23 CFS
10-YR Storm	N/A	0.18 CFS	0.17 CFS	0.35 CFS
100-YR Storm	N/A	0.33 CFS	0.26 CFS	0.59 CFS

*0.37 cfs/acre in the 50-year storm

The stormwater discharged from the detention pipe and the bypass runoff from the east and west driveways will be conveyed to the northwest corner of the site. The current plan is to install a lift station at this location, from which pumps will force it uphill to the south boundary of the site. The discharge would be released to the existing ditch in the NE 51st St. right-of-way.

Redmond stormwater regulations require that the pump system be:

- Sized for the 10-year peak flow rate
- Designed as a duplex system for backup
- Powered by an emergency generator with automatic starting and automatic disconnect from the power grid
- Equipped with an audible alarm for pump failure

The regulations also require a three-hour backup storage volume. The estimated volume of flow in a 10 year storm to the lift station from the detention pipe and the undetained driveways is approximately 2765 cubic feet. With space and setback limitations, the only location available for this backup storage is in the driveway on the east side of the building. A concrete vault will be constructed, measuring 8 ft. x 70 ft. x 5 ft. deep. The vault will be connected to the lift station by a 12-inch pipe, through which stormwater will back up into the vault in the event of pump failure. The vault will drain back by gravity when the pumps return to service. The elevation of the vault was set to ensure that stormwater will not overtop any catchbasins in the system when the vault is full.

Pump sizing: The lift station pumps have been preliminarily sized to manage the calculated 100-year flow released from the detention tank, together with the undetained flow from the paved bypass areas that are captured and routed to the lift station. This flow was conservatively estimated as 0.50 cfs (225 gpm), including slightly more than half of the 100-year flow from all of the bypass areas. Each of the two pumps will have a capacity of at least 125 gpm with 25 ft. total dynamic head. The pumps, controlled by float switches, will alternate for low flows and operate together to achieve maximum flow in large storms. Final sizing and selection of pumps will be completed when the site configuration is finalized.

Pump discharge: The stormwater pump is located in the northwest corner of the site. The pump will lift water and convey the metered runoff through a 4" forcemain to the east, to the northeast corner of the site. At this point, the forcemain will turn north and drain north through an existing public drainage easement. The forcemain will discharge into an 8" storm drain, where it will gravity drain the final 11 before it is discharged into a new catchbasin. The new catchbasin will be located on the existing storm drainage alignment.

WATER QUALITY DESIGN

Water quality treatment will be provided by StormFilter cartridge facilities. A five-cartridge facility in a 72-inch manhole will be located upstream from the detention tank to treat runoff from most of the paved area on the site. A separate single-cartridge catch basin installation will be located at the end of the driveway along the west side of the building, which provides resident/supplemental parking and vehicle access for deliveries and trash collection. Another separate single-cartridge catch basin installation will provide treatment for runoff from the east driveway, which provides only fire and maintenance access and will see minimal vehicular use. The two cartridge installations will provide treatment for pavement runoff that bypasses the detention tank. Discharge from both facilities will be tightlined to the lift station.

LID Feasibility Review

LID BMPs were reviewed to meet the objective of onsite stormwater management. These BMPs included:

Infiltration: Infiltration for this site has been reviewed and is not feasible. The Geotechnical Report, prepared by Robinson Nobel, Inc. and dated 03/11/2013, notes that the soils found onsite consist mainly of a Vashon Till. The soils were observed by two separate borings onsite, both of which had depths up to 6.5' deep. As reported in the geotechnical report, till soils contain a low infiltration rate. The site soils and low infiltration rate do not provide a reasonable site to implement infiltrating facilities for the proposed development. A copy of the Geotechnical Report is included in the Appendix to this report.

Dispersion: Dispersion for this site has been reviewed and is not feasible. Dispersion requires substantial amounts of undisturbed native vegetation on-site for a downstream flowpath. This site contains minimal native vegetation. In addition, the project site has been laid out and designed to meet the City of Redmond building setbacks, as noted per the City of Redmond zoning code. Following these setbacks, no space is left over for an onsite dispersion trench flowpath.

Pervious Pavement: Pervious Pavement has been reviewed and is not feasible. The Geotechnical Report, prepared by Robinson Nobel, Inc. and dated 03/11/2013, notes that the soils found onsite consist mainly of a Vashon Till. The soils were observed by two separate borings onsite, both of which had depths up to 6.5' deep. As reported in the geotechnical report, till soils contain a low infiltration rate. The site soils and low infiltration rate do not provide a reasonable site to implement infiltrating facilities for the proposed development. A copy of the Geotechnical Report is included in the Appendix to this report.

Amended Soils: Amended soils will be used for all new and replaced green areas.

SITE ASSESSMENT FOR LID

In accordance with Section 8.7.5 of the 2012 Stormwater Notebook, an LID site assessment has been completed for this project. This section provides responses to the following requirements:

1. A survey prepared by a registered land surveyor showing existing public and private development, including utility infrastructure, on and adjacent to the site, major and minor hydrologic features, including seeps, springs, closed depression areas, drainage swales, and 2 foot contours up to 10 percent slope and 5 foot contours for slopes above 10 percent. Spot elevations shall be at 25 foot intervals.
A survey has been prepared by GeoDimensions, dated 11/10/2015, and provides the information as noted in Item #1. A copy of the survey is included in the Appendix to this report.

2. Location of all existing lot lines, lease areas and easements.
A survey has been prepared by GeoDimensions, dated 11/10/2015, and provides the information as noted in Item #2. A copy of the survey is included in the Appendix to this report.

3. A soils report prepared by a licensed geotechnical engineer or licensed engineering geologist. The report shall identify:
    a. Underlying soils on the site utilizing soil pits and soil grain analysis to assess infiltration capability on site. The frequency and distribution of test pits shall be adequate to direct placement of the roads and structures away from soils that can most effectively infiltrate stormwater;
    b. Percolation tests if appropriate or requested by the Stormwater Engineer;
    c. Topographic and geologic features that may act as natural stormwater storage or conveyance and underlying soils that provide opportunities for storage and partial infiltration;
    d. Depth to wet season high groundwater;
    e. Geologic hazard areas and associated buffer requirements as defined in RZC 21.64.060;
    f. Distance from site boundaries to any areas within 200 feet of the site identified as landslide hazard areas or having a slope of 40 percent or steeper with a vertical relief of 10 feet or more; [Note: the City may require the applicant to expand the 200 feet to encompass a larger area if there are concerns for downstream geological hazards.]
    g. Identification of Wellhead Protection Zone(s); and
    h. For previously cleared or graded sites, analysis of topsoil according to the soil requirements in the City of Redmond Standard Specifications, Section 9.14.1.

A Geotechnical Report has been prepared by Robinson Nobel, Inc. dated 03/11/2013, and provides the information as noted in Item #3. A copy of the Geotechnical Report is included in the Appendix to this report.

4. A survey of existing native vegetation cover and wildlife habitat by a qualified biologist identifying any forest areas on the site, species and condition of ground cover and shrub layer, and tree species, seral stage, and canopy cover.
The existing site has minimal native vegetation cover and consists of a derelict house and a row of evergreen trees that line the driveway. The remainder of the site is largely covered with dense grass and blackberry bushes.

5. A streams, wetland, and water body survey and classification report by a qualified biologist showing wetland and buffer boundaries consistent with the requirements of RZC 21.64.030 and Critical Areas Reporting Requirements (RZC Appendix 1).
No stream, wetland, or body of water is located in or near the project site. A Critical Areas Map has been prepared by DCI Engineers using the King County iMap interactive mapping tool which maps streams, wetlands, and bodies of water. This Critical Areas Map is included in the Appendix to this report.

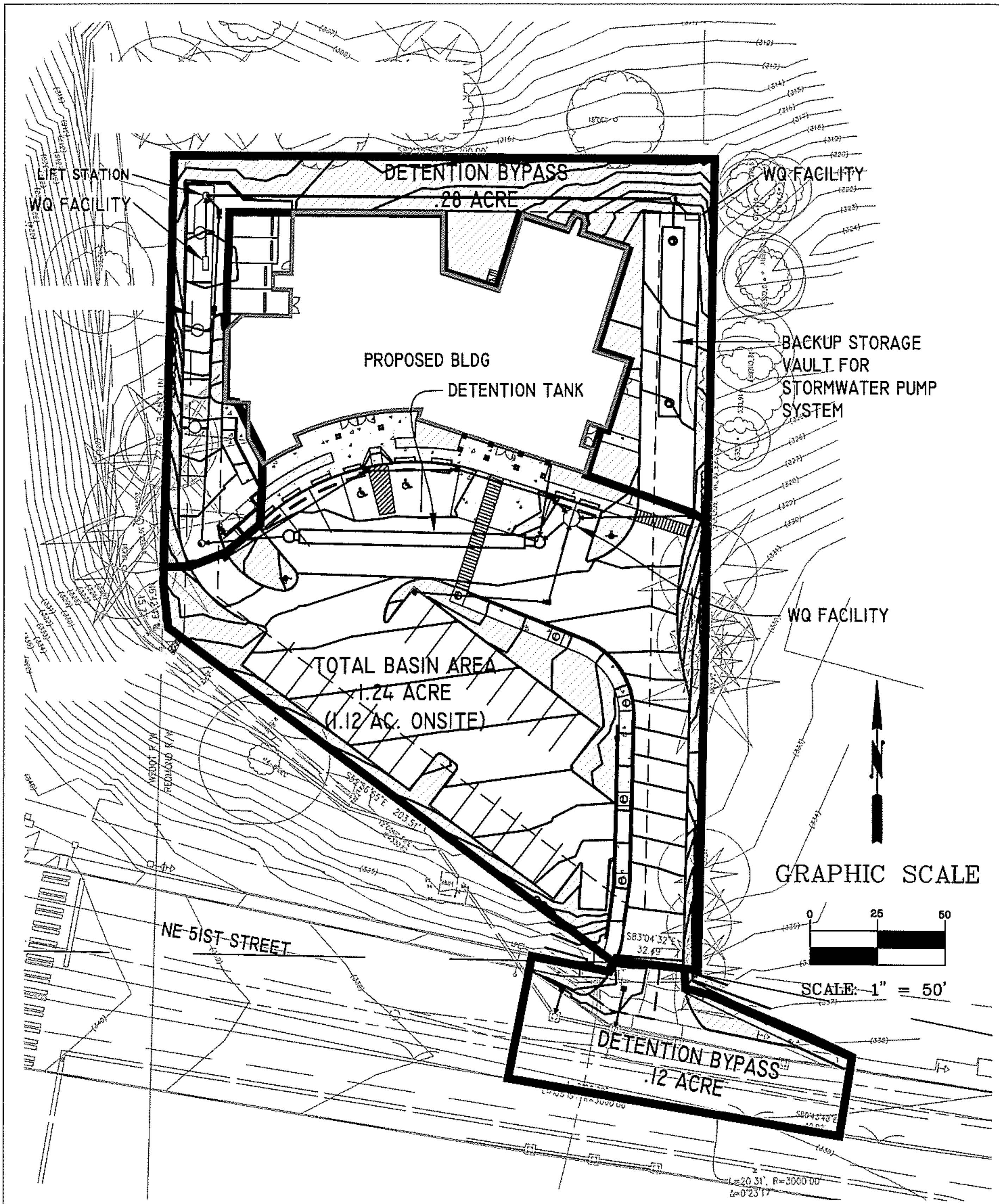
6. Flood hazard areas on or adjacent to the site.
This project site is not in or adjacent to a flood hazard area. A Critical Areas Map has been prepared by DCI Engineers using the King County iMap interactive mapping tool which maps flood hazard areas. This Critical Areas Map is included in the Appendix to this report.

7. A preliminary drainage report providing analysis of the existing site hydrologic conditions on the site and recommendations for type, location, and restrictions on LID BMPs.
The Site Assessment for LID is included in the preliminary drainage report.

8. Other studies as deemed necessary by the Stormwater Engineer.
No other studies are deemed necessary.

APPENDIX

Site & Basin Map
North Overlake Drainage Area Map
Water Quality Facility Sizing Calculations
Preliminary Detention Pipe Sizing Calculations
Preliminary Lift Station Backup Volume Calculation
Downstream Analysis Map
Topographic Survey
Critical Areas Map
Geotechnical Report



PROJECT NAME:	PROJECT NO:	BY: JWC	SHEET NO:
ANJUMAN E BURHANI	12012-0006	DATE: 11/1/13	1/1

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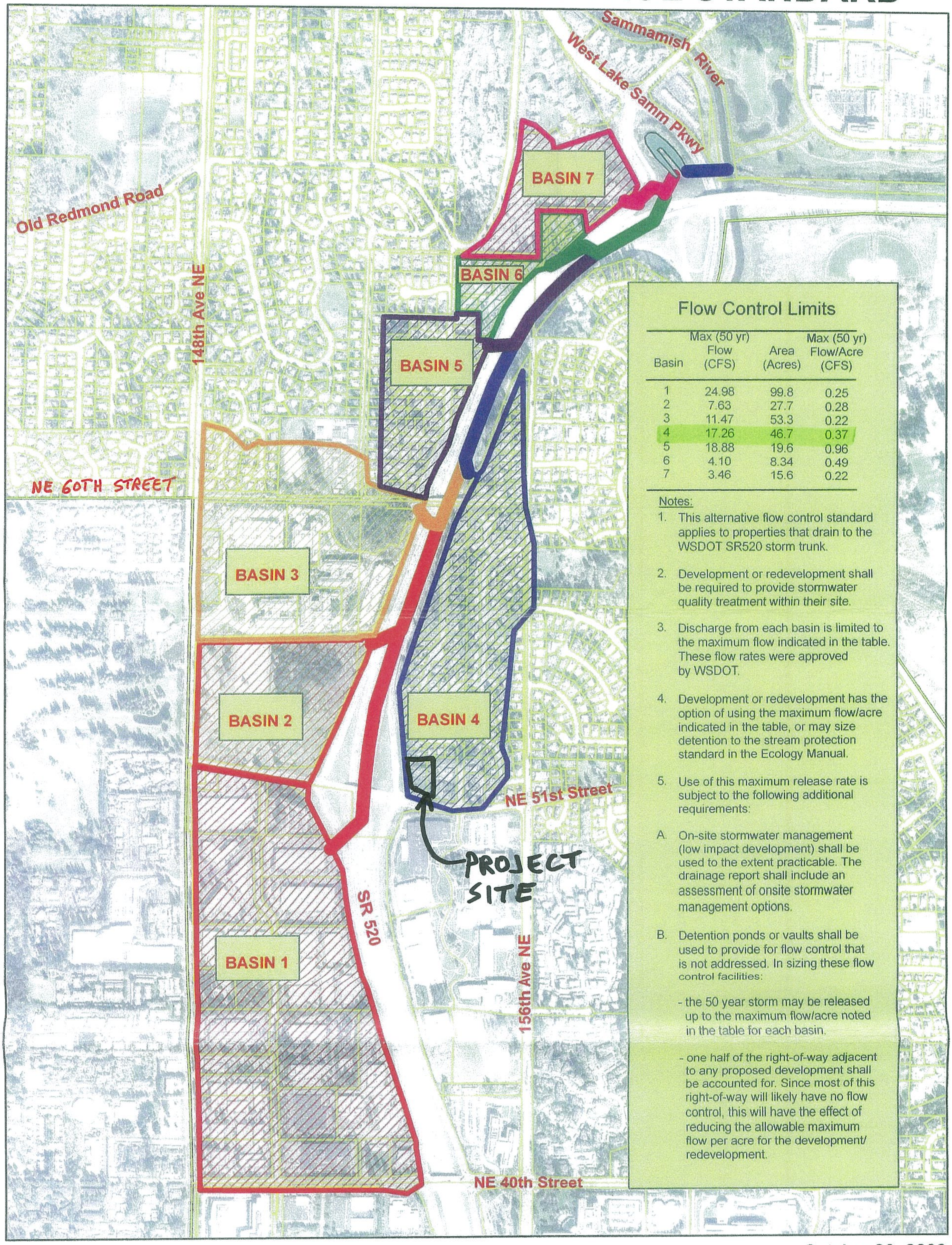
TITLE:

PRELIMINARY
SITE PLAN
& BASIN MAP

Basin Map.dwg 04 Nov 2013 – 9:00 am jcherry

CITY OF REDMOND PUBLIC WORKS
NORTH OVERLAKE DRAINAGE AREA
ALTERNATIVE FLOW CONTROL STANDARD



Flow Control Limits

Basin	Max (50 yr) Flow (CFS)	Area (Acres)	Max (50 yr) Flow/Acre (CFS)
1	24.98	99.8	0.25
2	7.63	27.7	0.28
3	11.47	53.3	0.22
4	17.26	46.7	0.37
5	18.88	19.6	0.96
6	4.10	8.34	0.49
7	3.46	15.6	0.22

Notes:

1. This alternative flow control standard applies to properties that drain to the WSDOT SR520 storm trunk.
2. Development or redevelopment shall be required to provide stormwater quality treatment within their site.
3. Discharge from each basin is limited to the maximum flow indicated in the table. These flow rates were approved by WSDOT.
4. Development or redevelopment has the option of using the maximum flow/acre indicated in the table, or may size detention to the stream protection standard in the Ecology Manual.
5. Use of this maximum release rate is subject to the following additional requirements:

A. On-site stormwater management (low impact development) shall be used to the extent practicable. The drainage report shall include an assessment of onsite stormwater management options.

B. Detention ponds or vaults shall be used to provide for flow control that is not addressed. In sizing these flow control facilities:

- the 50 year storm may be released up to the maximum flow/acre noted in the table for each basin.
- one half of the right-of-way adjacent to any proposed development shall be accounted for. Since most of this right-of-way will likely have no flow control, this will have the effect of reducing the allowable maximum flow per acre for the development/ redevelopment.

October 28, 2009

tyofRedmond

N W E S

0 360 720 1,440 2,160 2,880 Feet

Determining Number of Cartridges for Flow Based Systems

CONTECH Stormwater Solutions Inc. Engineer:	CRH
Date	7/31/2013
Site Information	
Project Name	Anjunani E Burhani
Project State	Washington
Project Location	Redmond
Basin ID	Main
Drainage Area, Ad	0.50 ac
Impervious Area, Ai	0.50 ac
Pervious Area, Ap	0.00
% Impervious	100%
Runoff Coefficient, Rc	0.95
Water quality flow	0.08 cfs
Peak storm flow	0.22 cfs
Filter System	
Filtration brand	StormFilter
Cartridge height	18 in
Specific Flow Rate	1.00 gpm/ft^2
Flow rate per cartridge	7.5 gpm

SUMMARY

Number of Cartridges	5

CONTECH ENGINEERED SOLUTIONS

Determining Number of Cartridges for Flow Based Systems

CONTECH Stormwater Solutions Inc. Engineer:	CRH	
Date	7/31/2013	

Site Information

Project Name	Anjunani E Burhani	
Project State	Washington	
Project Location	Redmond	
Basin ID	East	
Drainage Area, Ad	0.05	ac
Impervious Area, Ai	0.05	ac
Pervious Area, Ap	0.00	
% Impervious	100%	
Runoff Coefficient, Rc	0.95	
Water quality flow	0.01	cfs
Peak storm flow	0.02	cfs

Filter System

Filtration brand	StormFilter	
Cartridge height	18	in
Specific Flow Rate	1.00	gpm/ft^2
Flow rate per cartridge	7.5	gpm

SUMMARY

Number of Cartridges	1

Determining Number of Cartridges for Flow Based Systems

CONTECH Stormwater Solutions Inc. Engineer:	CRH	
Date	7/31/2013	
Site Information		
Project Name	Anjunani E Burhani	
Project State	Washington	
Project Location	Redmond	
Basin ID	West	
Drainage Area, Ad	0.07	ac
Impervious Area, Ai	0.07	ac
Pervious Area, Ap	0.00	
% Impervious	100%	
Runoff Coefficient, Rc	0.95	
Water quality flow	0.01	cfs
Peak storm flow	0.03	cfs
Filter System		
Filtration brand	StormFilter	
Cartridge height	18	in
Specific Flow Rate	1.00	gpm/ft^2
Flow rate per cartridge	7.5	gpm

SUMMARY

Number of Cartridges	1

Summary for Subcatchment 7S: Developed Site

Runoff = 0.62 cfs @ 7.93 hrs, Volume= 0.211 af, Depth> 3.02"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-24.00 hrs, dt= 0.05 hrs
Type IA 24-hr 50-YEAR STORM Rainfall=3.60"

	Area (ac)	CN	Description
*	0.700	98	Roof & pavement
*	0.140	74	Landscape
	0.840	94	Weighted Average
	0.140	74	16.67% Pervious Area
	0.700	98	83.33% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.3					**Direct Entry, Minimum**

Summary for Subcatchment 10S: BYPASS

Runoff = 0.24 cfs @ 7.95 hrs, Volume= 0.085 af, Depth> 2.54"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-24.00 hrs, dt= 0.05 hrs
Type IA 24-hr 50-YEAR STORM Rainfall=3.60"

	Area (ac)	CN	Description
	0.160	74	>75% Grass cover, Good, HSG C
*	0.240	98	Impervious
	0.400	88	Weighted Average
	0.160	74	40.00% Pervious Area
	0.240	98	60.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.3					**Direct Entry, MINIMUM**

Summary for Pond 9P: Detention Pipe

Inflow Area = 0.840 ac, 83.33% Impervious, Inflow Depth > 3.02" for 50-YEAR STORM event
Inflow = 0.62 cfs @ 7.93 hrs, Volume= 0.211 af
Outflow = 0.22 cfs @ 8.88 hrs, Volume= 0.209 af, Atten= 65%, Lag= 56.7 min
Primary = 0.22 cfs @ 8.88 hrs, Volume= 0.209 af

Routing by Stor-Ind method, Time Span= 0.00-24.00 hrs, dt= 0.05 hrs
Peak Elev= 104.78' @ 8.88 hrs Surf.Area= 0.005 ac Storage= 0.036 af

Plug-Flow detention time= 73.3 min calculated for 0.208 af (99% of inflow)
Center-of-Mass det. time= 63.9 min (740.2 - 676.3)

Volume	Invert	Avail.Storage	Storage Description
#1	100.00'	0.037 af	**60.0" D x 83.0'L Pipe Storage S= 0.0050 '/'**

Device	Routing	Invert	Outlet Devices
#1	Primary	100.50'	**2.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#2	Primary	104.80'	**0.5' long Sharp-Crested Rectangular Weir** 2 End Contraction(s)
#3	Primary	105.00'	**8.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.22 cfs @ 8.88 hrs HW=104.78' (Free Discharge)
—**1=Orifice/Grate** (Orifice Controls 0.22 cfs @ 9.96 fps)
—**2=Sharp-Crested Rectangular Weir** (Controls 0.00 cfs)
—**3=Orifice/Grate** (Controls 0.00 cfs)

Summary for Subcatchment 7S: Developed Site

Runoff = 0.66 cfs @ 7.93 hrs, Volume= 0.224 af, Depth> 3.21"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-24.00 hrs, dt= 0.05 hrs
Type IA 24-hr 100-YEAR STORM Rainfall=3.80"

	Area (ac)	CN	Description
*	0.700	98	Roof & pavement
*	0.140	74	Landscape
	0.840	94	Weighted Average
	0.140	74	16.67% Pervious Area
	0.700	98	83.33% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.3					**Direct Entry, Minimum**

Summary for Subcatchment 10S: BYPASS

Runoff = 0.26 cfs @ 7.95 hrs, Volume= 0.090 af, Depth> 2.71"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-24.00 hrs, dt= 0.05 hrs
Type IA 24-hr 100-YEAR STORM Rainfall=3.80"

	Area (ac)	CN	Description
	0.160	74	>75% Grass cover, Good, HSG C
*	0.240	98	Impervious
	0.400	88	Weighted Average
	0.160	74	40.00% Pervious Area
	0.240	98	60.00% Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.3					**Direct Entry, MINIMUM**

Summary for Pond 9P: Detention Pipe

Inflow Area = 0.840 ac, 83.33% Impervious, Inflow Depth > 3.21" for 100-YEAR STORM event
Inflow = 0.66 cfs @ 7.93 hrs, Volume= 0.224 af
Outflow = 0.33 cfs @ 8.37 hrs, Volume= 0.222 af, Atten= 50%, Lag= 26.3 min
Primary = 0.33 cfs @ 8.37 hrs, Volume= 0.222 af

Routing by Stor-Ind method, Time Span= 0.00-24.00 hrs, dt= 0.05 hrs
Peak Elev= 104.97' @ 8.37 hrs Surf.Area= 0.004 ac Storage= 0.037 af

Plug-Flow detention time= 74.3 min calculated for 0.222 af (99% of inflow)
Center-of-Mass det. time= 64.6 min (739.7 - 675.1)

Volume	Invert	Avail.Storage	Storage Description
#1	100.00'	0.037 af	**60.0" D x 83.0'L Pipe Storage S= 0.0050 '/'**

Device	Routing	Invert	Outlet Devices
#1	Primary	100.50'	**2.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads
#2	Primary	104.80'	**0.5' long Sharp-Crested Rectangular Weir** 2 End Contraction(s)
#3	Primary	105.00'	**8.0" Horiz. Orifice/Grate** C= 0.600 Limited to weir flow at low heads

Primary OutFlow Max=0.32 cfs @ 8.37 hrs HW=104.96' (Free Discharge)
├─**1=Orifice/Grate** (Orifice Controls 0.22 cfs @ 10.17 fps)
├─**2=Sharp-Crested Rectangular Weir** (Weir Controls 0.10 cfs @ 1.31 fps)
└─**3=Orifice/Grate** (Controls 0.00 cfs)

Events for Pond 9P: Detention Pipe

Event	Inflow (cfs)	Primary (cfs)	Elevation (feet)	Storage (acre-feet)
2 YR STORM	0.29	0.13	102.02	0.012
10 YR STORM	0.47	0.18	103.30	0.024
50-YEAR STORM	0.62	0.22	104.78	0.036
100-YEAR STORM	**0.66**	**0.33**	**104.97**	**0.037**

Events for Subcatchment 10S: BYPASS

Event	Runoff (cfs)	Volume (acre-feet)	Depth (inches)
2 YR STORM	0.10	0.036	1.09
10 YR STORM	0.17	0.062	1.87
50-YEAR STORM	0.24	0.085	2.54
100-YEAR STORM	**0.26**	**0.090**	**2.71**

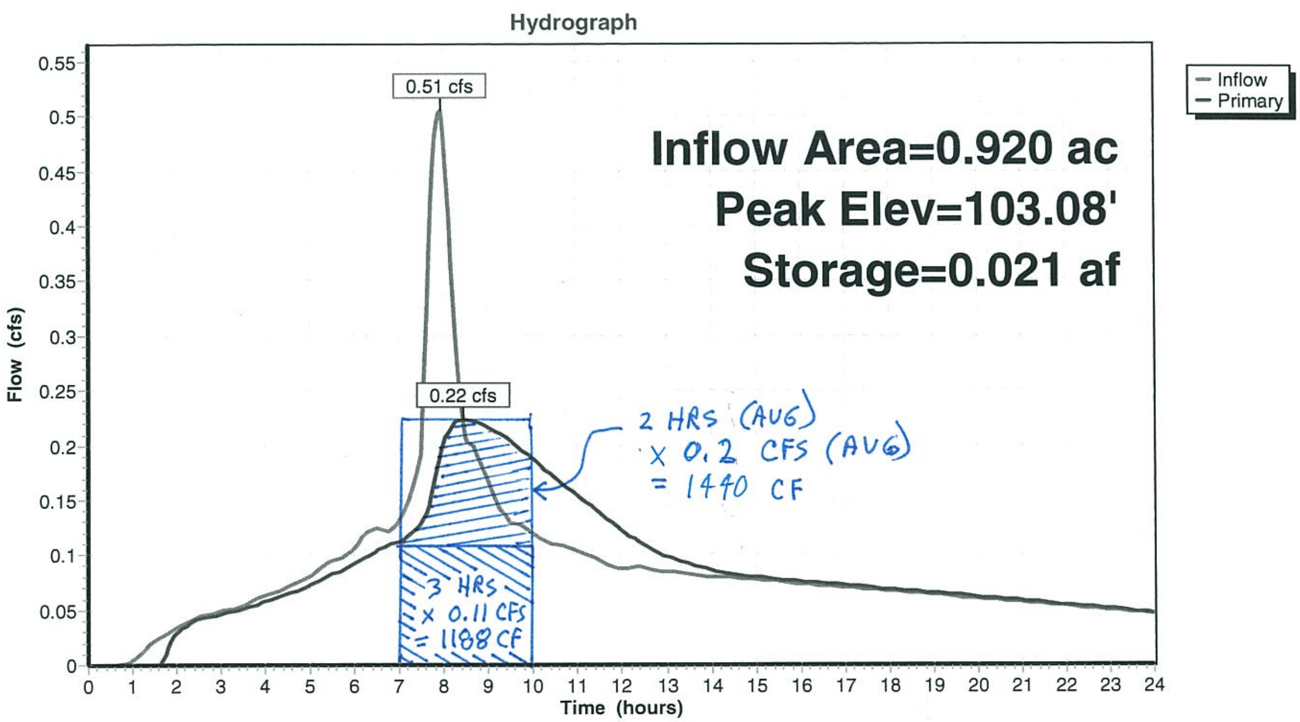
PRELIMINARY LIFT STATION BACKUP VOLUME CALCULATION

The hydrographs on the following pages provide a rough approximation of the volume that would be required for three-hour back-up storage for the tentatively proposed stormwater lift station. The area under the curve during the peak three hours of a 10-year storm represents the volume of stormwater that is handled by the pumps during that three hour period.

The total volume is the combination of the discharge from the detention pipe and the flow from that portion of the bypass area that drains to the lift station, approximately 20% of the total bypass area. The estimated total volume is 2764 cu. ft.

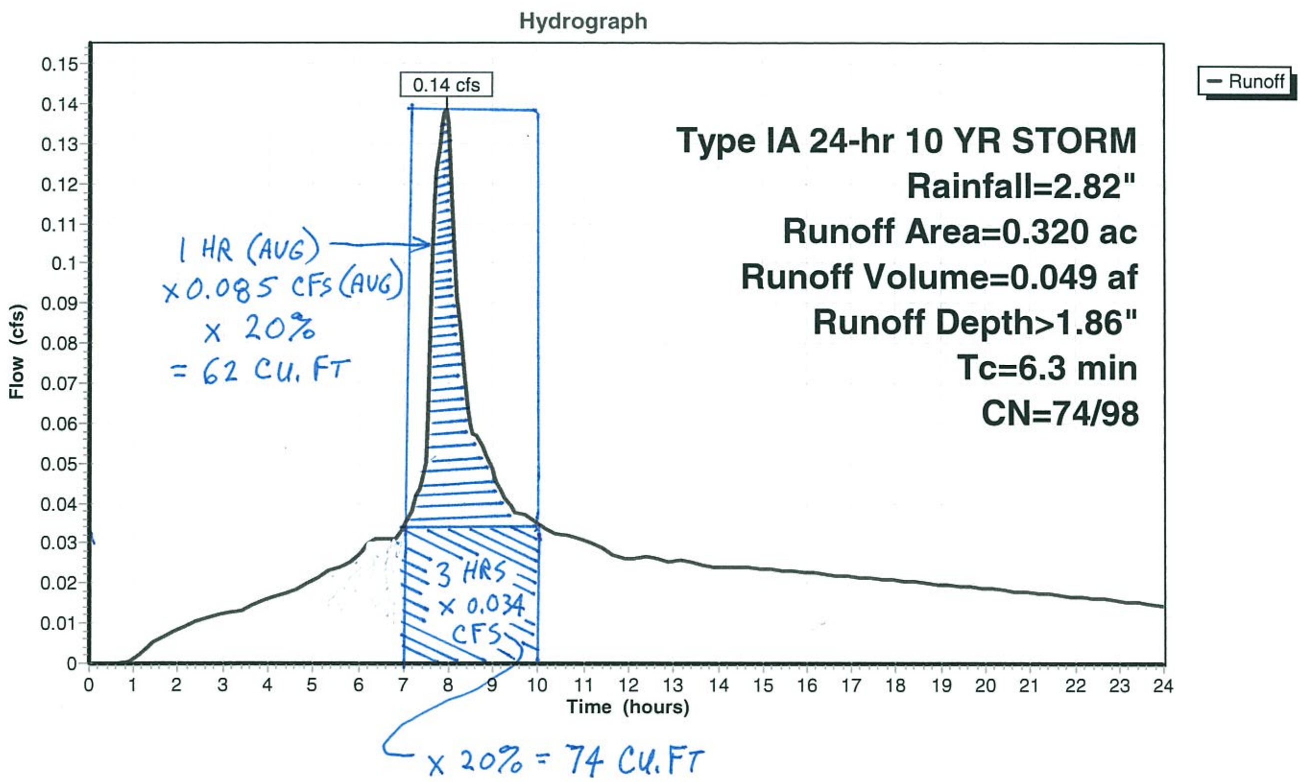
A more precise sizing analysis will be provided for the final design if the lift station remains as part of the drainage system.

Pond 9P: Detention Pipe



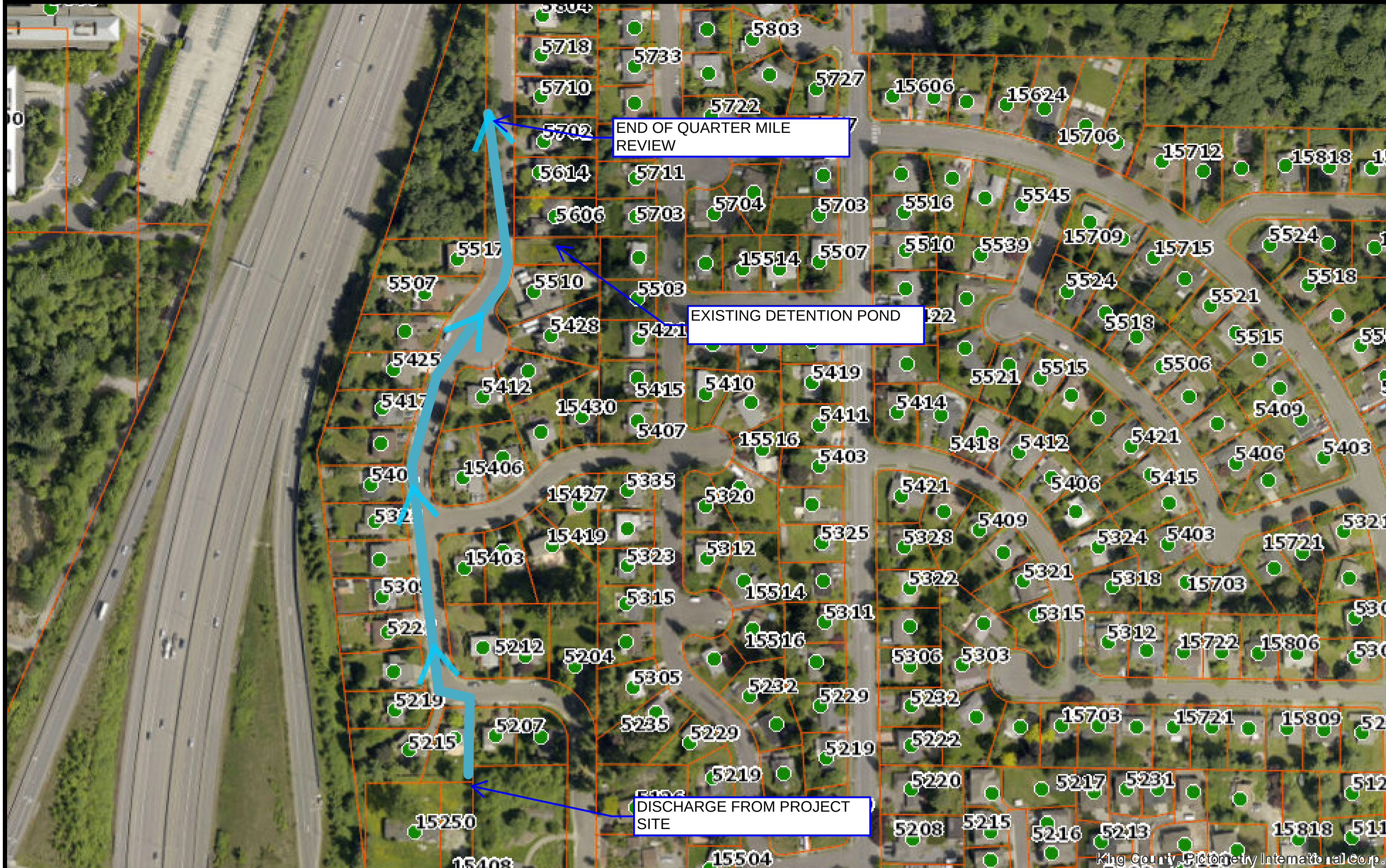
TOTAL VOLUME: 2628 CU. FT. FOR THE DISCHARGE FROM THE DETENTION PIPE

Subcatchment 10S: BYPASS



TOTAL VOLUME: 136 CU. FT. FOR THE 20%
OF THE BYPASS AREA THAT DRAINS
TO THE LIFT STATION

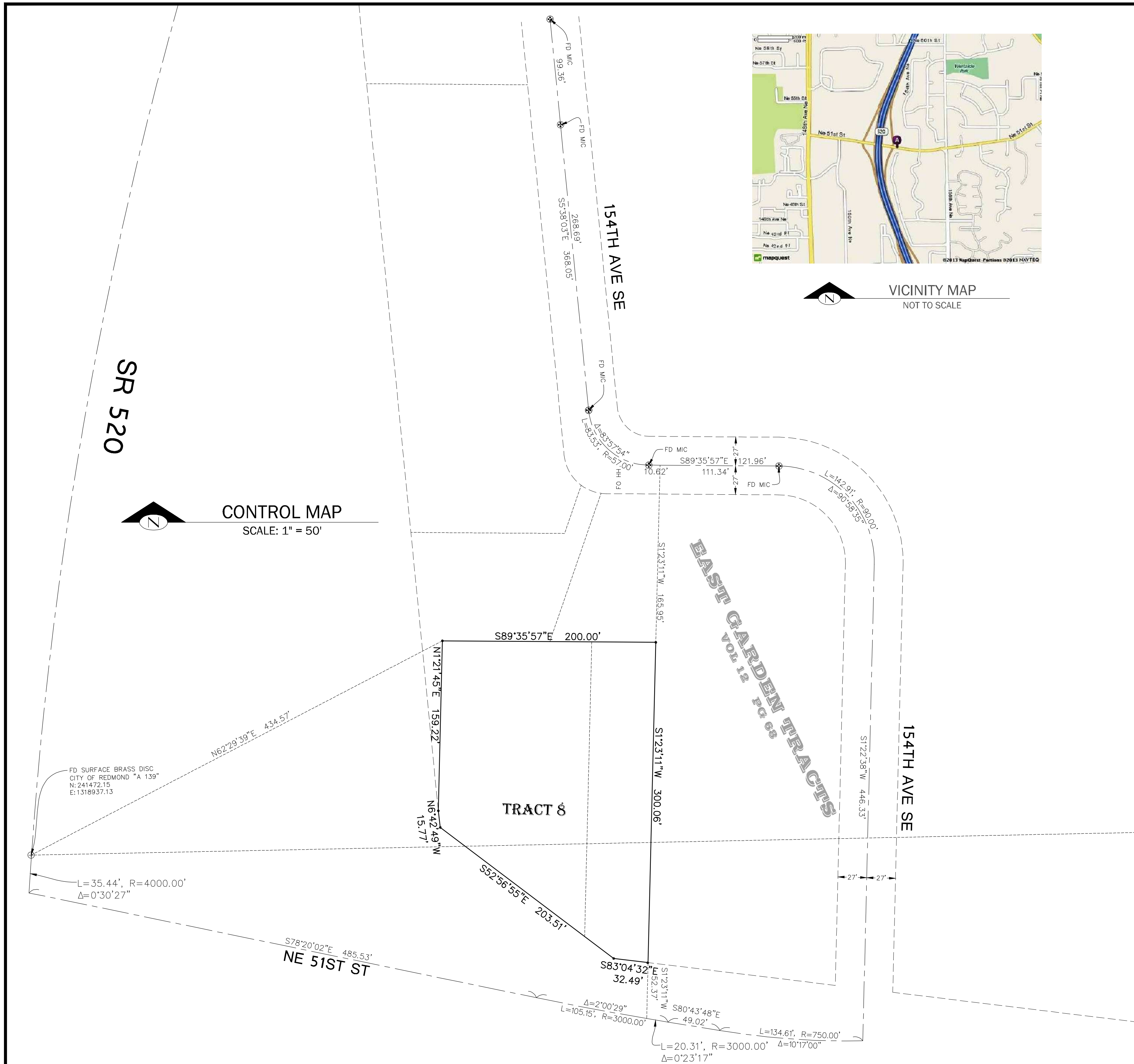
Downstream Drainage Map



Date: 3/31/2016 Notes:

N

King County
GIS Center



SURVEY LEGEND

ASPH	ASPHALTIC CONCRETE
	BUILDING LINE
C	CONCRETE CURB
CLF	CHAIN LINK FENCE
CONC	CONCRETE SURFACE
CRW	CONCRETE RETAINING WALL
CW	CONCRETE WALK
	EASEMENT AREA
	FOUND SURVEY MONUMENT
	FENCE LINE (CHAIN LINK)
GM	GAS METER
HH	HAND HOLE
HYD	FIRE HYDRANT
IP	IRON PIPE
LP	LIGHT POLE
LUM	LUMINAIRE
MH	MAINTENANCE HOLE
PM	POWER METER
PP	POWER POLE
PP	POWER POLE W/ LIGHT
	SIGN
	TRAFFIC SIGNAL
THH	TELEPHONE HAND HOLE
WM	WATER METER
	VALVE
	FENCE LINE (WOOD)
YL	YARD LIGHT

SITE ADDRESS:
15250 AND 15252 NORTHEAST 51ST STREET
REDMOND, WASHINGTON 98052

TAX PARCEL NUMBER(S):
15250 NORTHEAST 51ST STREET = 218250-0082-06
15252 NORTHEAST 51ST STREET = 218250-0080-08.

ZONING:
R-5 SINGLE-FAMILY URBAN RESIDENTIAL

FLOOD MAP:
LOCATED IN ZONE "X" AND IS OUTSIDE 500 YEAR FLOODPLAIN PER FLOOD INSURANCE RATE MAP NUMBER 53033C0369F, DATED MAY 16, 1995.

AREA:
TOTAL SITE AREA IS 48,978.65 SQUARE FEET OR 1.1244 ACRES, MORE OR LESS.

METHOD OF SURVEY:
INSTRUMENTATION FOR THIS SURVEY WAS A LEICA TOTAL STATION UNIT. PROCEDURES USED IN THIS SURVEY WERE DIRECT AND REVERSE ANGLES, NO CORRECTION NECESSARY. MEETS WASHINGTON STATE STANDARDS SET BY WAC 332-130-090.

UNDERGROUND UTILITIES:
BURIED UTILITIES SHOWN BASED ON RECORDS FURNISHED BY OTHERS AND VERIFIED WHERE POSSIBLE IN THE FIELD. GEODIMENSIONS ASSUMES NO LIABILITY FOR THE ACCURACY OF THOSE RECORDS OR ACCEPT RESPONSIBILITY FOR UNDERGROUND LINES WHICH ARE NOT MADE PUBLIC RECORD. FOR THE FINAL LOCATION OF EXISTING UTILITIES IN AREAS CRITICAL TO DESIGN CONTACT THE UTILITY OWNER/AGENCY. AS ALWAYS, CALL 1-800-424-5555 BEFORE CONSTRUCTION.

VERTICAL DATUM: NAVD88

BENCHMARKS:
CITY OF REDMOND ID# COR 9129
ELEV=359.03'
PUNCH MARK IN 3" DIAMETER BRASS DISC IN NORTHWEST CORNER OF CONCRETE PAD FOR TRAFFIC POLE AT THE NORTHEAST QUADRANT OF BRIDLE TRAIL CROSSING (WHERE NE 60TH ST WOULD BE IF CONSTRUCTED) AND 148TH AVE. NE. STAMPED "CITY OF REDMOND BM 57".

CITY OF REDMOND ID# COR 9130
ELEV=261.50'
2" DIAMETER BRASS DISC SET IN THE SOUTHEAST CORNER OF CONCRETE POWER VAULT IN THE NORTHWEST QUADRANT OF THE INTERSECTION OF THE NE 60TH ST. AND 156TH AVE. NE. STAMPED "CITY OF REDMOND 9130, 2009"

HORIZONTAL DATUM: NAD83(91)

BASIS OF BEARING: LINE BETWEEN CITY OF REDMOND HORIZONTAL CONTROL MONUMENTS "A 139" AND "A 144" BEARING N88°49'38"E

LEGAL DESCRIPTION:

TRACT 8 OF EAST SIDE GARDENS, ACCORDING TO THE PLAT THEREOF RECORDED IN VOLUME 12 OF PLATS, PAGE 68, RECORDS OF KING COUNTY, WASHINGTON;
EXCEPT THE NORTH 1011.34 FEET THEREOF;
AND EXCEPT THE EAST 290 FEET THEREOF;
AND EXCEPT THAT PORTION THEREOF CONVEYED TO KING COUNTY FOR ROAD BY DEED RECORDED UNDER RECORDING NO. 2716295;
AND EXCEPT THAT PORTION THEREOF CONDEMNED IN SUPERIOR COURT CAUSE NO. 750450 FOR S.R. 520;

SITUATE IN KING COUNTY, WASHINGTON.

REFERENCE:
LEGAL DESCRIPTION BASED ON STATUTORY WARRANTY DEED FURNISHED BY CHICAGO TITLE, RECORDED IN KING COUNTY RECORDS UNDER INSTRUMENT NUMBER, 20100813000869 DATED AUGUST 13, 2010.

Edwin J. Green Jr.
EDWIN J. GREEN JR. PLS#15025

8/6/2015
DATE

GeoDimensions
GeoDimensions, Inc., 35402 SE Center Street, Snoqualmie, WA 98065
phone 425.458.4488 support@geodimensions.net
www.geodimensions.net

EDWIN J. GREEN, JR. STATE OF WASHINGTON 15025 REGISTERED PROFESSIONAL LAND SURVEYOR

NO	REVISION	DATE
1	ADD'L TOPO OFFSITE TO NORTH & 154TH AVE (TLR)	8/6/15

TOPOGRAPHIC SURVEY
SE 1/4 OF NW 1/4 OF SEC 14, TWP 25N, RGE 5E, WM
BURHANI PROJECT
15252 NE 51ST ST
SEATTLE WASHINGTON

JOB NO.: 8211
DATE: 4/11/2013
DRAFTED BY: CJC
CHECKED BY: EJG
SCALE: 1" = 20'

GeoDimensions

GeoDimensions, Inc., 35402 SE Center Street, Snoqualmie, WA 98065
phone 425.458.4488 support@geodimensions.net
www.geodimensions.net

NO	REVISION	DATE
3	UTILITIES MARKED BY APS & LOCATED ON 11/9/15 (TLR)	11/10/15
2	ADD 10' EASEMENT PER PLAT	8/19/15
1	ADD'L TOPO OFFSITE TO NORTH & 154TH AVE (TLR)	8/6/15

TOPOGRAPHIC SURVEY
SE 1/4 OF NW 1/4 OF SEC 14, TWP 25N, RGE 5E, WM

BURHANI PROJECT

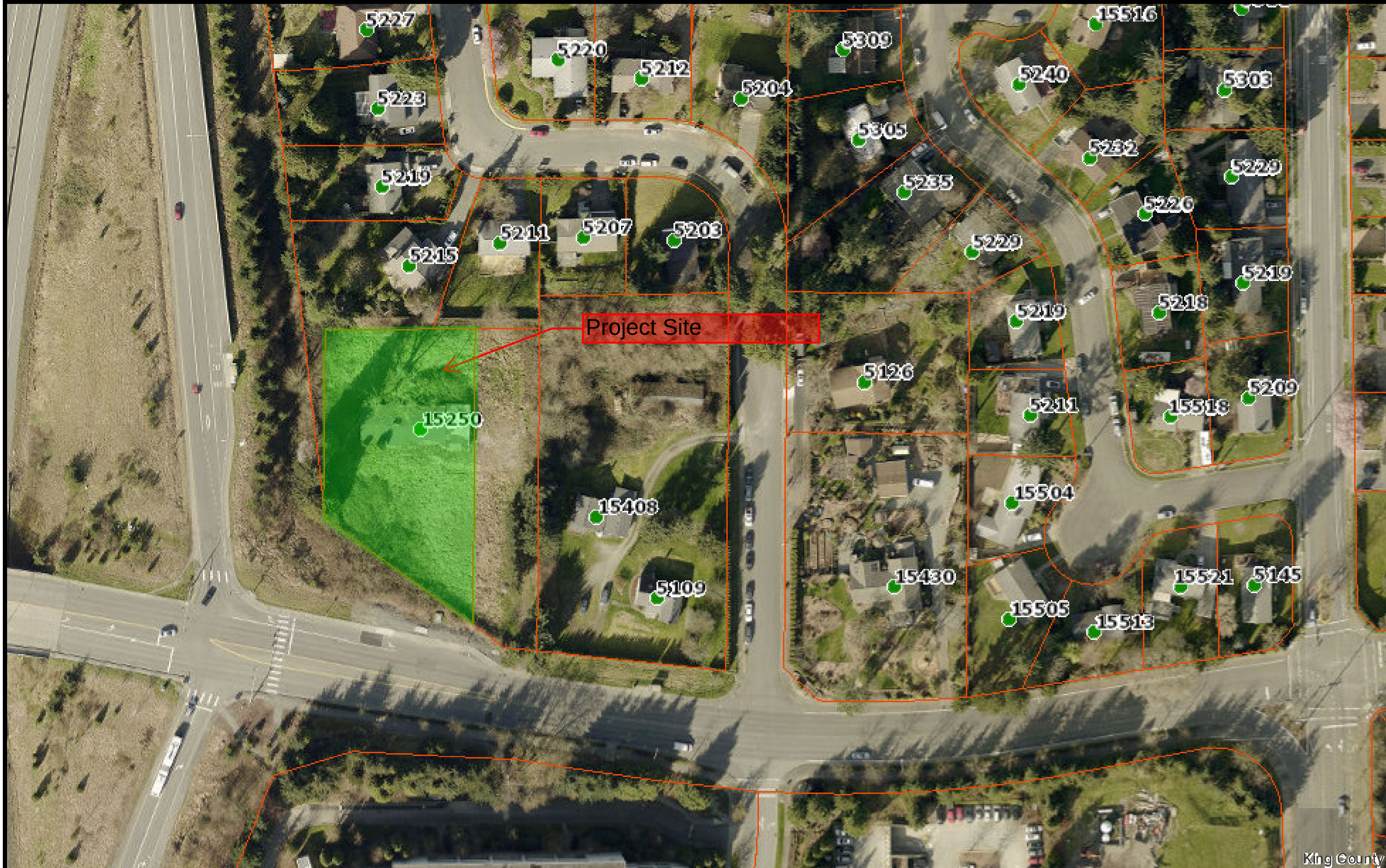
15252 NE 51ST ST

SEATTLE WASHINGTON

JOB NO.: 8211
DATE: 3/7/2013
DRAFTED BY: CJC
CHECKED BY: EJG
SCALE: 1" = 20'

2 OF 2

AeB-Critical Areas Map



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Date: 10/6/2016

Notes:

NO CRITICAL AREAS WITHIN VICINITY OF PROJECT

N

King County
GIS CENTER

ROBINSON
NOBLE

March 11, 2013

Mr. Samuel E Cameron
Rolluda Architects
105 South Main Street, Suite 323
Seattle, Washington 98104

Geotechnical Engineering Report
Anjuman E Burhani Community Complex
Redmond, Washington
RN File No. 2791-001A

Dear Mr. Cameron:

This letter transmits three copies of our report for the Anjuman E Burhani Community Complex located at 15252 NE 51st Street in Redmond, Washington. The subsurface soils encountered are capable of providing support for the planned building and pavement.

We appreciate the opportunity of working with you on this project. If you have any questions regarding this report, please contact us.

Sincerely,

Rick B Powell, PE
Principal Engineer

KHB:BAG:RBP:am

Three Copies Submitted
Eight Figures

3011 South Huson Street, Suite A
Tacoma, Washington 98409
P: 253.475.7711 | F: 253.472.5846

www.robinson-noble.com

17625 130th Avenue NE, Suite 102
Woodinville, Washington 98072
P: 425.488.0599 | F: 425.488.2330

TABLE OF CONTENTS

INTRODUCTION

This report presents the results of our geotechnical engineering investigation at the proposed community complex project in King County, Washington. The site is located at the intersection of NE 51st Street and the north-bound on-ramp for SR 520, as shown on the Vicinity Map in Figure 1.

You have requested that we complete this geotechnical report to evaluate subsurface conditions and provide recommendations for site development.

PROJECT DESCRIPTION

The project will consist of the construction of a new three story 20,725 square foot community complex with associated access drive isles and parking at the intersection of Northeast 51st Street and State Route 520 in Redmond. We understand the structure will be constructed in a two-phase approach. Phase 1 will include the construction of a new multi-purpose facility and Phase 2 will include the construction of a new mosque.

SCOPE

The scope of services to be provided by Robinson Noble, Inc. is for geotechnical evaluation services, including the following:

- Review available geologic maps for the site.
- Explore the subsurface soil and groundwater conditions with a subcontracted drill rig.
- Evaluate pertinent physical and engineering characteristics of the soils encountered in the borings.
- Prepare a geotechnical report with our conclusions and recommendations for geotechnical design elements of the project. Our report will include:
    - Description of the geologic materials.
    - Depth to groundwater encountered during drilling.
    - Evaluation of infiltration feasibility.
    - Discussion of seismicity at the site along with seismic design parameters including Site Class and site coefficients based on current IBC criteria.
    - Excavation considerations and temporary slope angles.
    - Recommendations for shallow foundations including allowable soil bearing values, minimum footing sizes, soil parameters for lateral load resistance, and footing drains.
    - Estimate the total and differential settlements of spread footings and floor slabs for variable loading within the building.
    - Geotechnical recommendations and considerations for support of concrete slab-on-grade floors and sub-slab drainage.
    - Recommendations for parking and drive isle subgrade preparation and design considerations.
    - Recommendations for earthwork and site preparation. An evaluation of the effects of weather and/or construction equipment on site soils and mitigation of any unsuitable soil.

- Comment on any anticipated construction difficulties identified from the results of our site studies and from our experience on projects at similar sites.

SITE CONDITIONS

Surface Conditions

The project site is about 2 acres in size and has maximum dimensions of approximately 220 feet in the east-west direction and 310 feet in the north-south direction. Access to the site is provided by NE 51st Street to the south. The site is also bordered by existing residential acreage to the east and north, and the on-ramp for north-bound SR 520 to the west. A layout of the site is shown on the Site Plan in Figure 2.

The site slopes gently down to the northwest. A single-family residence with outbuildings currently sits within the site. The site is vegetated mostly with grass, blackberry bushes, landscaping shrubs, and contains a few small- to- medium sized trees.

Geology

Most of the Puget Sound Region was affected by past intrusion of continental glaciation. The last period of glaciation, the Vashon Stade of the Fraser Glaciation, ended approximately 14,000 years ago. Many of the geomorphic features seen today are a result of scouring and overriding by glacial ice. During the Vashon Stade, areas of the Puget Sound region were overridden by over 3,000 feet of ice. Soil layers overridden by the ice sheet were compacted to a much greater extent than those that were not. Part of a typical glacial sequence within the area of the site includes the following soil deposits from newest to oldest:

> **Artificial Fill (af)** – Fill material is often locally placed by human activities, consistency will depend on the source of the fill. The thickness and expanse of this material will be dependent on extent of fill required to grade land to the desired elevations. Density of the fill will depend on earthwork activities and compaction efforts made during the placement of the material.
>
> **Vashon Till (Qvt)** – The till is a non-sorted mixture of clay, sand, pebbles, cobbles and boulders, all in variable amounts. The till was deposited directly by the ice as it advanced over and eroded irregular surfaces of previously deposited formations and sediments. The till was well compacted by the advancing glacier and exhibits high strength and stability. Drainage is considered very poor in the till.

The geologic units for this area are mapped on the Geologic Map of King County, Washington, by Derek B. Booth and Aaron P. Wisher (U.S. Geological Survey, February 2006). The site is mapped as being underlain by a deposit of glacial till. Our site explorations encountered glacial drift. Glacial drift is similar to glacial till, but may exhibit more sorting of various soil grain sizes.

Explorations

We explored subsurface conditions within the site on February 22, 2013, by drilling three borings with a portable hollow stem auger drill rig. The borings were drilled to depths of 16.5 to 21.5 feet below the ground surface. Samples were obtained from the borings at 2.5 and 5-foot intervals by driving a split spoon sampler with a 140-pound hammer dropping 30 inches. The number of blows required for penetration of three 6-inch intervals was recorded. To determine the standard penetration number at that depth the number of blows required for the lower two intervals are summed. If the number of blows reached 50 before the sampler was driven through any 6-inch interval, the sampler was not driven further and the blow count is recorded as 50 for the actual penetration distance.

The borings were located in the field by an engineer from this firm who also examined the soils and geologic conditions encountered, and maintained logs of the borings. The approximate locations of the borings are shown on the Site Plan in Figure 2. The soils were visually classified in general accordance with the Unified Soil Classification System, a copy of which is presented as Figure 3. The logs of the borings are presented in Figures 4 through 6.

Subsurface Conditions

A brief description of the conditions encountered in our explorations is included below. For a more detailed description of the soils encountered, review the Boring Logs in Figures 4 through 6.

Our explorations generally encountered a layer of soft and loose sandy silt to silty sand that was approximately 4 feet in thickness. Underlying the sandy silt to silty sand we encountered soils interpreted as glacial drift. These soils consisted of medium dense silty fine to medium sand in Boring 1, hard sandy silt in Boring 2 and very dense silty fine to medium sand in Boring 3 to depths of about 8 feet below ground surface (bgs). This was underlain in all explorations by dense to very dense fine to medium sand with varying amounts of silt and gravel to the depths explored of 16.5 to 21.5 feet.

Hydrologic Conditions

Groundwater was encountered in Boring 2 at 19 feet bgs and in Boring 3 at 15 feet bgs. The groundwater appears to be a perched condition within the sandy portions of the glacial drift. Due to the elevation of the site and the geologic conditions, we do not expect the groundwater levels are part of a regional groundwater table.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion that the site is compatible with the planned development. The underlying medium dense to very dense glacial drift deposits are capable of supporting the planned structures and pavements. We recommend that the foundations for the structures extend through any fill, topsoil, loose, or disturbed soils, and bear on the underlying medium dense or firmer native glacial drift, or on structural fill extending to these soils. Based on our site explorations, we anticipate these soils will generally be encountered at depths ranging from 4 to 5.5 feet.

Geologic Hazards

Erosion Hazard: The erosion hazard criteria used for determination of affected areas includes soil type, slope gradient, vegetation cover, and groundwater conditions. The erosion sensitivity is related to vegetative cover and the specific surface soil types (group classification), which are related to the underlying geologic soil units. We reviewed the Web Soil Survey by the Natural Resources Conservation Service (NRCS) to determine the erosion hazard of the on-site soils. The site surface soils were classified using the SCS classification system as Arents, Alderwood material (AmB) with 0 to 6 percent slope. The corresponding geologic unit for these soils is till, which is in agreement with the soils encountered in our site explorations. The erosion hazard for the soil is listed as being slight for the gently sloping conditions.

Seismic Hazard: It is our opinion based on our subsurface explorations that the Soil Profile in accordance with the 2009 and 2012 International Building Code (IBC) is Site Class C with Seismic Design Category D. We used the US Geological Survey program "U.S. Seismic Design Maps Web Application." The design maps summary reports for the 2009 and 2012 IBC are included in this report as Appendix A.

Additional seismic considerations include liquefaction potential and amplification of ground motions by soft soil deposits. The liquefaction potential is highest for loose sand with a high groundwater table. The underlying dense to very dense glacial drift soils are considered to have a very low potential for liquefaction and amplification of ground motion.

Site Preparation and Grading

The first step of site preparation should be to strip the vegetation, topsoil, or loose soils to expose at least medium dense or stiff native soils in pavement and building areas. The excavated material should be removed from the site, or stockpiled for later use as landscaping fill. The resulting subgrade should be compacted to a firm, non-yielding condition. Areas observed to pump or yield should be repaired prior to placing hard surfaces.

The on-site glacial drift likely to be exposed during construction is considered highly moisture sensitive, and the surface will disturb easily when wet. We expect these soils would be difficult, if not impossible, to compact to structural fill specifications in wet weather. We recommend that earthwork be conducted during the drier months. Additional expenses of wet weather or winter construction could include extra excavation and use of imported fill or rock spalls. During wet weather, alternative site preparation methods may be necessary. These methods may include utilizing a smooth-bucket trackhoe to complete site stripping and diverting construction traffic around prepared subgrades. Disturbance to the prepared subgrade may be minimized by placing a blanket of rock spalls or imported sand and gravel in traffic and roadway areas. Cutoff drains or ditches can also be helpful in reducing grading costs during the wet season. These methods can be evaluated at the time of construction.

Structural Fill

General: All fill placed beneath buildings, pavements or other settlement sensitive features should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is observed by an experienced geotechnical professional or soils technician. Field observation procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction.

Materials: Imported structural fill should consist of a good quality, free-draining granular soil, free of organics and other deleterious material, and be well graded to a maximum size of about 3 inches. Imported, all-weather structural fill should contain no more than 5 percent fines (soil finer than a Standard U.S. No. 200 sieve), based on that fraction passing the U.S. 3/4-inch sieve.

The use of on-site soil as structural fill will be dependent on moisture content control. Some drying of the native soils may be necessary in order to achieve compaction. During warm, sunny days this could be accomplished by spreading the material in thin lifts and compacting. Some aeration and/or addition of moisture may also be necessary. We expect that compaction of the native soils to structural fill specifications would be difficult, if not impossible, during wet weather.

Fill Placement: Following subgrade preparation, placement of the structural fill may proceed. Fill should be placed in 8- to 10-inch-thick uniform lifts, and each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill underlying building areas, and within a depth of 2 feet below pavement and sidewalk subgrade, should be compacted to at least 95 percent of its maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D1557 compaction test procedure. Fill more than 2 feet beneath sidewalks and pavement subgrades should be compacted to at least 90 percent of the maximum dry density. The moisture content of the soil to be compacted should be within about 2 percent of optimum so that a readily compactable condition exists. It may be necessary to overexcavate and remove wet surficial soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction.

Temporary and Permanent Slopes

Temporary cut slope stability is a function of many factors, such as the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable temporary cut slope geometry. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations, since the contractor is continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered.

For planning purposes, we recommend that temporary cuts in the near-surface weathered soils be no steeper than 1.5 Horizontal to 1 Vertical (1.5H:1V). Temporary cuts in the dense glacial drift should be no steeper than 1H:1V. If groundwater seepage is encountered, we would expect that flatter inclinations would be necessary.

We recommend that cut slopes be protected from erosion. Measures taken may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than 4 feet, if worker access is necessary. We recommend that cut slope heights and inclinations conform to local and WISHA/OSHA standards.

Final slope inclinations for granular structural fill and the native soils should be no steeper than 2H:1V. Lightly compacted fills, common fills, or structural fill predominately consisting of fine grained soils should be no steeper than 3H:1V. Common fills are defined as fill material with some organics that are "trackrolled" into place. They would not meet the compaction specification of structural fill. Final slopes should be vegetated and covered with straw or jute netting. The vegetation should be maintained until it is established.

Foundations

Conventional shallow spread foundations should be founded on at least undisturbed, medium dense or stiff soil. If the soil at the planned bottom of footing elevation is not suitable, it should be overexcavated to expose suitable bearing soil. Our explorations encountered soils suitable for bearing at depths ranging from 4 to 5.5 feet bgs. The footings could be supported on prisms of structural fill that extend down to native bearing soil. The excavation should extend laterally ½ the width of the footing on each side of the footing. Footings should extend at least 18 inches below the lowest adjacent finished ground surface for frost protection. Minimum foundation widths should conform to IBC requirements. Standing water should not be allowed to accumulate in footing trenches. All loose or disturbed soil should be removed from the foundation excavation prior to placing concrete.

For foundations constructed as outlined above, we recommend an allowable design bearing pressure as shown in Table 1 be used for the footing design. IBC guidelines should be followed when considering short-term transitory wind or seismic loads. Potential foundation settlement using the recommended allowable bearing pressure is estimated to be less than 1-inch total and ½-inch differential between footings or across a distance of about 30 feet. Higher soil bearing values may be appropriate with wider footings. These higher values can be determined after a review of a specific design.

Footing Type	Footing Width/dimensions (ft)	Bearing Capacity (psf)	Estimated Total settlement (inches)	Estimated Differential Settlement (inches)
Continuous	1.5	2500	1.0	0.5
Continuous	2	3200	1.0	0.5
Continuous	3	3800	1.0	0.5
Square	2.5 x 2.5	2700	1.0	0.5
Square	4 x 4	3300	1.0	0.5
Square	5 x 5	3800	1.0	0.5

Lateral Loads

The lateral earth pressure acting on retaining walls is dependent on the nature and density of the soil behind the wall, the amount of lateral wall movement, which can occur as backfill is placed, and the inclination of the backfill. Walls that are free to yield at least one-thousandth of the height of the wall are in an "active" condition. Walls restrained from movement by stiffness or bracing are in an "at-rest" condition. Active earth pressure and at-rest earth pressure can be calculated based on equivalent fluid density. Equivalent fluid densities for active and at-rest earth pressure of 35 pounds per cubic foot (pcf) and 55 pcf, respectively, may be used for design for a level backslope. These values assume that the on-site soils or imported granular fill are used for backfill, and that the wall backfill is drained. The preceding values do not include the effects of surcharges, such as due to foundation loads or other surface loads. Surcharge effects should be considered where appropriate. The above drained active and at-rest values should be increased by a uniform pressure of 6.8H and 18.9H psf, respectively, when considering seismic conditions using 2009 IBC. The above drained active and at-rest values should be increased by a uniform pressure of 6.7H and 18.7H psf, respectively, when considering seismic conditions using 2012 IBC H represents the wall height.

The above lateral pressures may be resisted by friction at the base of the wall and passive resistance against the foundation. A coefficient of friction of 0.42 may be used to determine the base friction in the native glacial drift soils. An equivalent fluid density of 270 pcf may be used for passive resistance design. To achieve this value of passive pressure, the foundations should be poured "neat" against the native dense soils, or compacted fill should be used as backfill against the front of the footing, and the soil in front of the wall should extend a horizontal distance at least equal to three times the foundation depth. A factor of safety of 1.5 has been applied to the passive pressure to account for required movements to generate these pressures. The friction coefficient does not include a factor of safety.

All wall backfill should be well compacted. Care should be taken to prevent the buildup of excess lateral soil pressures due to overcompaction of the wall backfill.

Slabs-On-Grade

Slab-on-grade areas should be prepared as recommended in the **Site Preparation and Grading** subsection. Slabs should be supported on at least medium dense or stiff native soils, or on structural fill extending to these soils. The subgrade modulus as prepared should have at least 150 pounds per cubic inch (pci).

Where moisture control is a concern, we recommend that slabs be underlain by 6 inches of pea gravel for use as a capillary break. A suitable vapor barrier, such as heavy plastic sheeting, should be placed over the capillary break. An additional 2-inch-thick damp sand blanket can be used to cover the vapor barrier to protect the membrane and to aid in curing the concrete. This will also help prevent cement paste bleeding down into the capillary break through joints or tears in the vapor barrier. The capillary break material should be connected to the footing drains to provide positive drainage.

Infiltration

We understand that infiltration is being considered for this project and will be dependent on allowable space within the lot with respect to parking constraints.

We completed two grain size distribution tests in general accordance with ASTM D422 in the area of the anticipated infiltration zone. Grain size distribution tests indicated that material observed between 5 and 6.5 feet was sandy silt in Boring 2 and was silty sand in Boring B-3. Results of the grain size distribution tests are included in this report as Figures 7 and 8.

A long-term infiltration rate was determined based on Figure 3.28, Infiltration Rate as a Function of the D10 Size of the Soil for Ponds in Western Washington from the Department of Ecology Stormwater Management Manual of Western Washington. A long-term infiltration rate of 0.2 inches per hour may be used to size the infiltration gallery in the sandy silt disclosed in the Boring 2 area, and 0.5 inches per hour may be used to size the infiltration gallery in the silty sand revealed in the vicinity of Boring 3. Additional explorations and infiltration tests will be required on site per the City of Redmond code if it is determined that an infiltration system may be used in the project design.

Drainage

We recommend that runoff from impervious surfaces, such as roofs, driveway and access roadways, be collected and routed to an appropriate stormwater discharge system. The finished ground surface should be sloped at a gradient of 5 percent minimum for a distance of at least 10 feet away from the buildings, or to an approved method of diverting water from the foundation. Surface water should be collected by permanent catch basins and drain lines, and be discharged into a storm drain system.

We recommend that footing drains be used around all of the structures where moisture control is important. It is good practice to use footing drains installed at least 1 foot below the planned finished floor slab elevation to provide drainage. Footing drains should consist of 4-inch-diameter, perforated PVC pipe that is surrounded by free-draining material, such as pea gravel. Footing drains should discharge into tightlines leading to an appropriate collection and discharge point. For slabs-on-grade, a drainage path should be provided from the capillary break material to the footing drain system. Roof drains should not be connected to wall or footing drains.

Our experience with gently-sloping glacial drift sites is that the volume of water collected by residence foundation drains and routed to the stormwater detention system is insignificant when considered in the storm drainage design. We do not expect that the foundation drain water will impact the design of the stormwater detention system.

Detention Pond

If a stormwater detention pond is planned, it should be excavated into the underlying native soils. We recommend that any fill berms be constructed of soils having a maximum permeability of 1×10^{-5} centimeters per second (4×10^{-6} inches/second). The on-site sandy silt encountered in Boring 2 meets this criterion. We should evaluate any proposed berm fill material prior to construction of the berm.

If a pond is to be constructed, the cut slopes of the pond should be no steeper than 3H:1V on the inside of the detention pond and no steeper than 2H:1V above the water table or on the outside portions of the pond berms. Inside slopes as steep as 2H:1V are possible but may require maintenance until vegetation is established. Areas with seepage may require a blanket of rock spalls or other measures to limit sloughing.

Where any berms for the pond are to be constructed, the topsoil and loose soils should be removed down to the competent glacial drift. Areas to receive new fill should be stripped of unsuitable surface soils and compacted to a firm, non-yielding state prior to placement of the new fill. The excavation should be kept dry to allow the proper placement of structural fill. Structural fill should be placed and compacted as discussed in the **Structural Fill** subsection of this report. We recommend that the fill in any pond berms be compacted to a minimum of 92 percent of its maximum dry density as determined by the ASTM D1557 compaction test procedure. After each lift of the fill in a berm is compacted to specification, the surface should be scarified to a depth of 2 inches prior to placement of the next lift. The purpose of the scarification is to reduce the risk of creating preferential seepage paths through the pond or berms.

It will be important to compact the face of any pond fill embankments. This should be made explicit to the contractor performing the on-site work. Uncompacted soils on a berm face will be more susceptible to erosion and sloughing. If groundwater seepage is encountered within a cut slope face, a layer of rock spalls may be necessary to minimize erosion of the slope face. The spall layer can be placed at the time of construction, or in the future if sloughing of the slope is observed.

Detention Vault

If a stormwater detention vault is planned, the concrete walls of the vault may be supported on footing foundations bearing on the underlying competent glacial drift. We recommend a soil bearing pressure of 4000 pounds per square foot (psf) for the design of the wall footings poured on undisturbed competent glacial drift.

We recommend that footing drains be installed on the outside of perimeter footings. The footing drains should be at least 4 inches in diameter and should consist of perforated or slotted, rigid, smooth-walled PVC pipe, laid at the bottom of the footings. The drain line should be surrounded with free-draining pea gravel or coarse sand and wrapped with a layer of non-woven filter fabric. A vertical drainage blanket at least 12 inches thick, consisting of compacted pea gravel or other free-draining granular soils, should be placed against the walls. A vertical drain mat, such as Miradrain 6000 by Mirafi Inc., may be placed against the walls in lieu of the vertical drainage blanket. Structural fill is then placed behind the vertical drainage blanket or drain mat to backfill the walls. The vertical drainage blanket or drain mat should be hydraulically connected to the drain line at the base of the walls. Sufficient number of cleanouts at strategic locations should be installed for periodical cleaning of the wall drain line to prevent clogging.

The perimeter walls of the concrete vault with a lid would be restrained at their top from horizontal movement and should be designed for at-rest lateral soil pressure, while the perimeter walls of a vault without a lid would be unrestrained at the top and may be designed for active lateral soil pressure. Active earth pressure and at rest earth pressure can be calculated based on equivalent fluid density. Equivalent fluid densities for active and at rest earth pressure of 35 pcf and 55 pcf, respectively, may be used for design for a level backslope. These values assume that the on-site soils are used for backfill, and that the wall backfill is drained. The preceding values do not include the effects of surcharges due to foundation loads, traffic or other surface loads. Surcharge effects should be considered where appropriate. Recommended seismic lateral loading is provided in the **Lateral Load** section of this report. For undrained soil conditions, the active and at-rest pressures should be increased to 80 pcf and 90 pcf, respectively. Undrained conditions may occur in the lower portion of the vault if there is not suitable fall to place a wall drain at the footing elevation.

All wall backfill should be well compacted. Care should be taken to prevent the buildup of excess lateral soil pressures due to overcompaction of the wall backfill.

Utilities

Our explorations indicate that deep dewatering will not be needed to install standard depth utilities. Anticipated groundwater is expected to be handled with pumps in the trenches. We also expect that some groundwater seepage may develop during and following the wetter times of the year. We expect this seepage to mostly occur in pockets. We do not expect significant volumes of water in these excavations.

The soils likely to be exposed in utility trenches after site stripping are considered highly moisture sensitive. We recommend that they be considered for trench backfill during the drier portions of the year. Provided these soils are within 2 percent of their optimum moisture content, they should be suitable to meet compaction specifications. During the wet season, it may be difficult to achieve compaction specifications; therefore, soil amendment with kiln dust or cement may be needed to achieve proper compaction with the on-site materials.

Pavement Subgrade

The performance of roadway pavement is critically related to the conditions of the underlying subgrade. We recommend that the subgrade soils within the roadways be prepared as described in the **Site Preparation and Grading** subsection of this report. Prior to placing base material, the subgrade soils should be compacted to a non-yielding state with a vibratory roller compactor and then proof-rolled with a piece of heavy construction equipment, such as a fully-loaded dump truck. Any areas with excessive weaving or flexing should be overexcavated and recompacted or replaced with a structural fill or crushed rock placed and compacted in accordance with recommendations provided in the **Structural Fill** subsection of this report.

We recommend that parking areas be designed with at least 2 inches of class B asphalt, underlain by 6 inches of crushed rock. Traffic areas for the complex should be designed with at least 3 inches of asphalt and 8 inches of crushed rock.

CONSTRUCTION OBSERVATION

We should be retained to provide observation and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, and to provide recommendations for design changes, should the conditions revealed during the work differ from those anticipated. As part of our services, we would also evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications.

USE OF THIS REPORT

We have prepared this report for Rolluda Architects and its agents, for use in planning and design of this project. The data and report should be provided to prospective contractors for their bidding and estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of subsurface conditions.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report, for consideration in design. There are possible variations in subsurface conditions. We recommend that project planning include contingencies in budget and schedule, should areas be found with conditions that vary from those described in this report.

Within the limitations of scope, schedule and budget for our services, we have strived to take care that our services have been completed in accordance with generally accepted practices followed in this area at the time this report was prepared. No other conditions, expressed or implied, should be understood.

We appreciate the opportunity to be of service to you. If there are any questions concerning this report or if we can provide additional services, please call.

Sincerely,

Robinson Noble, Inc.

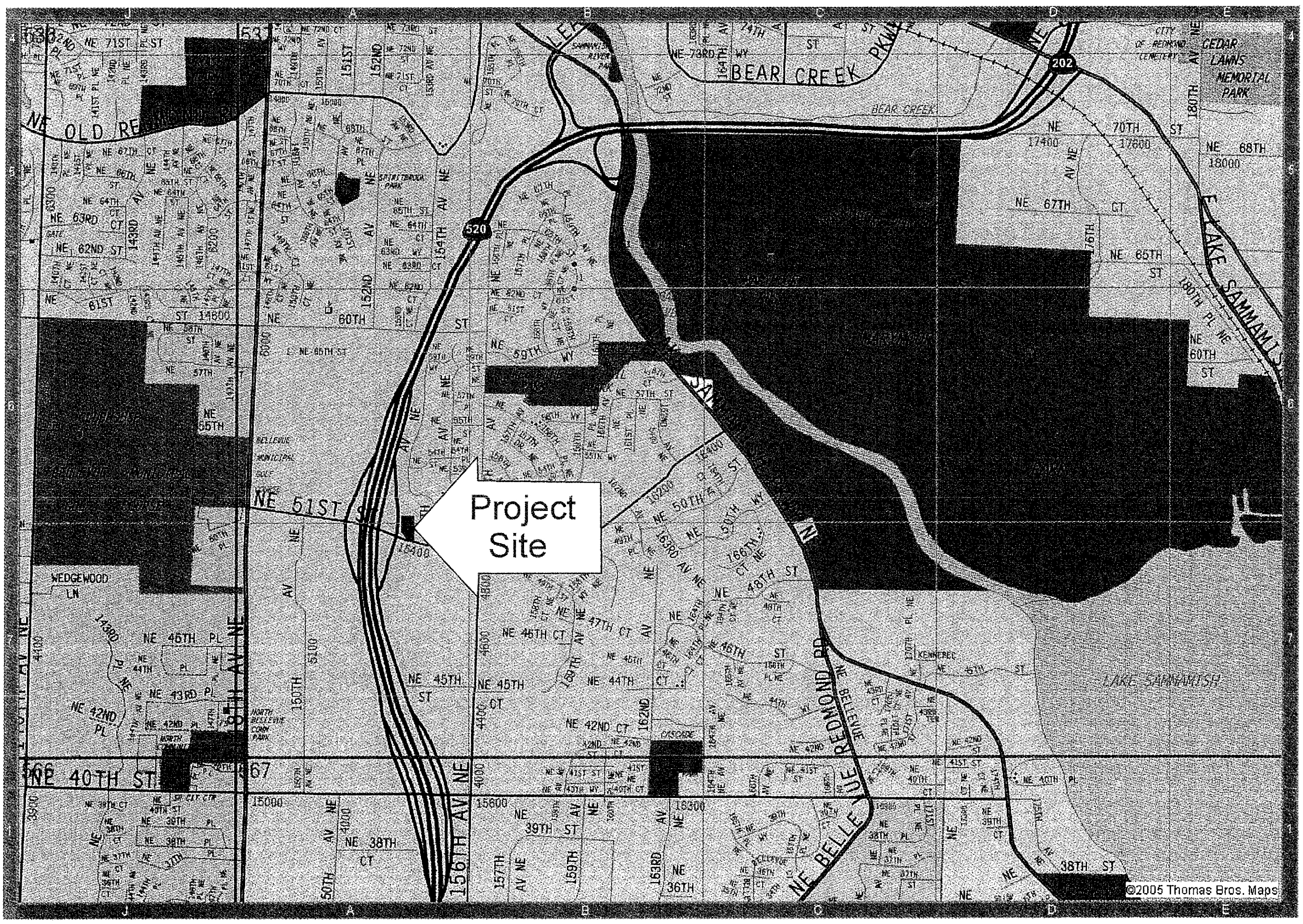
RICK B. POWELL
STATE OF WASHINGTON
30530
REGISTERED
PROFESSIONAL ENGINEER

3/11/2013

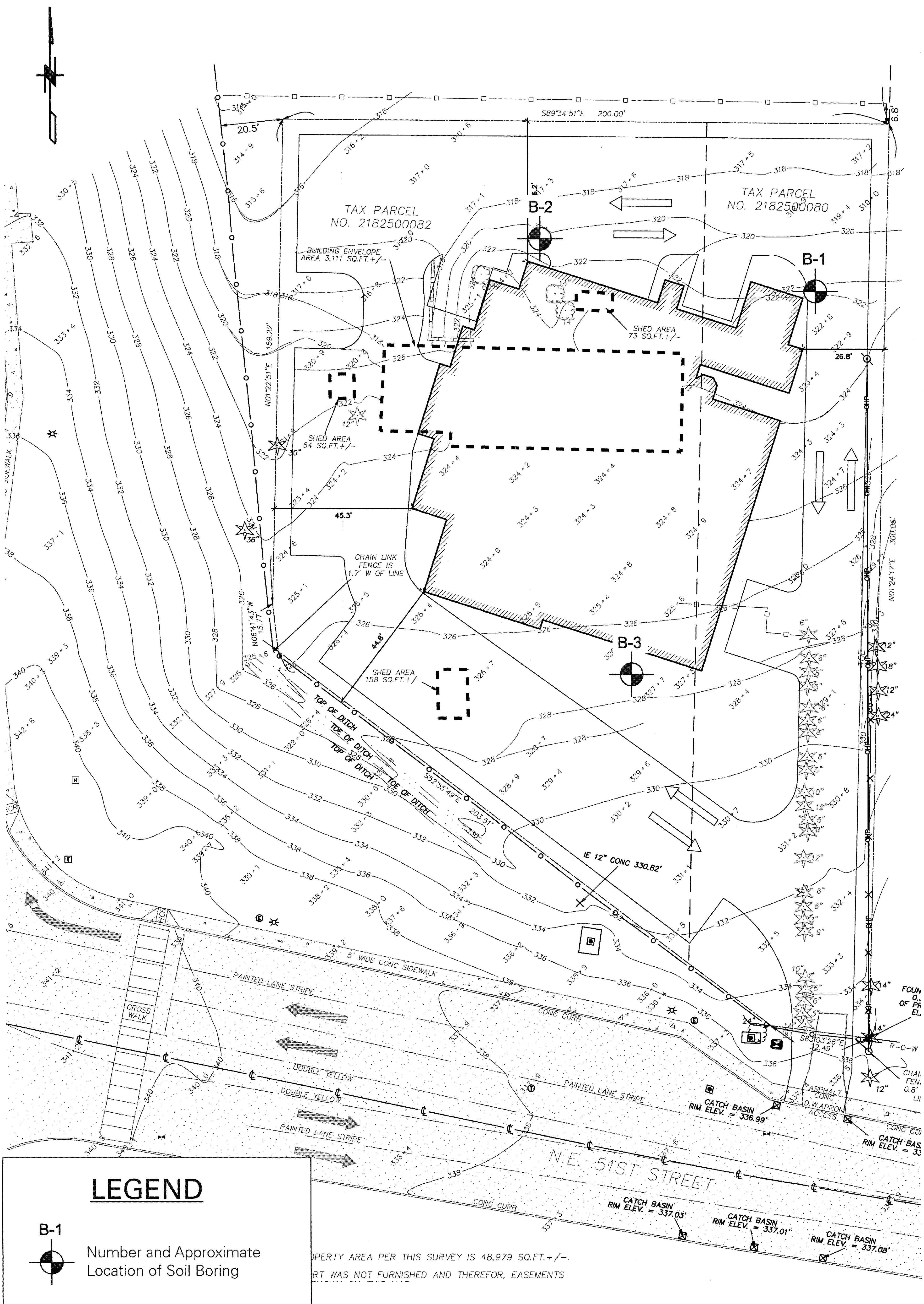
Rick B. Powell, PE
Principal Engineer

KHB:BAG:RBP:am

Three Copies Submitted
Eight Figures



ROBINSON NOBLE		PM: RBP March 2013 2791-001A	Figure 1 Vicinity Map Rolluda Architects: Anjuman-E-Burhani Community Complex



LEGEND

B-1 Number and Approximate Location of Soil Boring

0' 30'

Approximate Scale

...OPERTY AREA PER THIS SURVEY IS 48,979 SQ.FT.+/-.
...RT WAS NOT FURNISHED AND THEREFOR, EASEMENTS

ROBINSON NOBLE	Note: Basemap taken from Site Survey prepared by Rolluda Architects dated 12/20/2011.	PM: RBP March 2013 2791-001A	Figure 2 Site Plan Rolluda Architects: Anjuman-E-Burhani Community Complex

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50% OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50% PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50%	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50% OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

* 1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-93.

* 2) Soil classification using laboratory tests is based on ASTM D 2487-93.

3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance, of soils, and/or test data.

* Modifications have been applied to ASTM methods to describe sit and clay content.

SOIL MOISTURE MODIFIERS

Dry- Absence of moisture, dusty, dry to the touch

Moist- Damp, but no visible water

Wet- Visible free water or saturated, usually soil is obtained from below water table

KEY TO BORING LOG SYMBOLS

SM — Letter symbol for soil type

— Contact between soil strata (Dashed line indicates approximate contact between soils)

ML — Letter symbol for soil type

∇ Ground water level

● Blows required to drive sample 12 in. using SPT

MC (■) = % Moisture = $\frac{\text{(Weight of water)}}{\text{(Weight of dry soil)}}$

DD = Dry Density

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual

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PM: RBP
February 2013
2791-001A

Figure 3

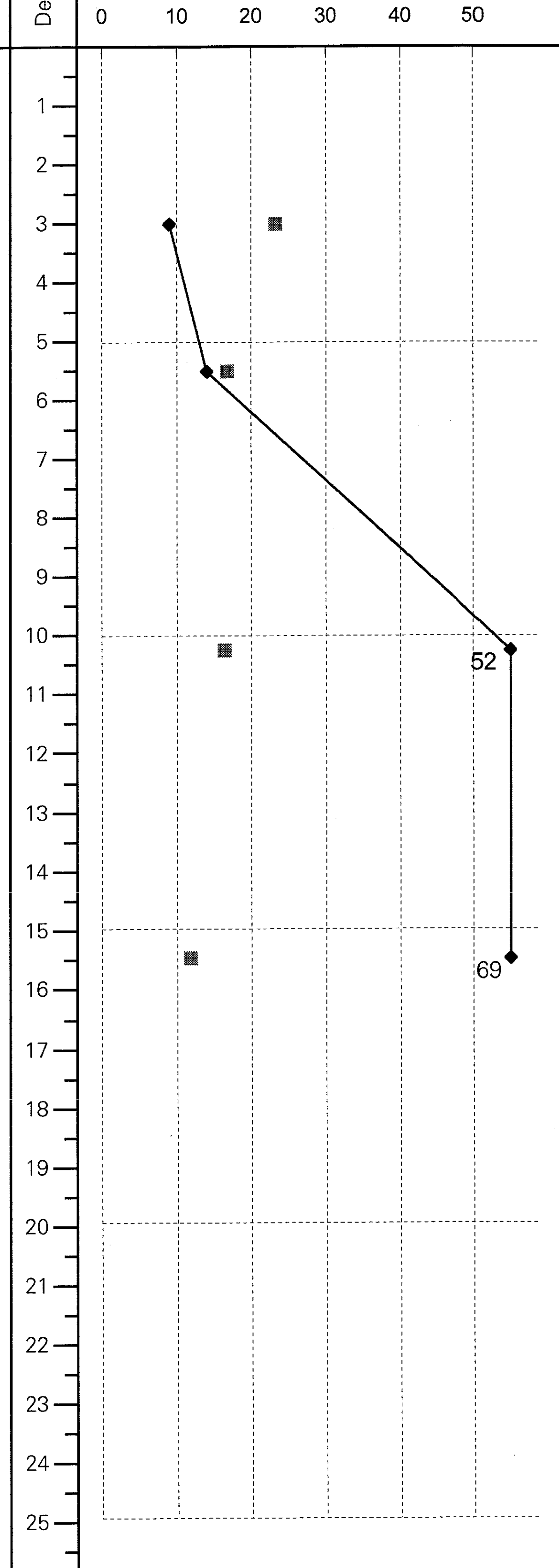
Rolluda Architects: Anjuman-E-Burhani Community Complex

B-1			
Date	2/22/2013	Hole diameter	
Logged by	KHB	Hole depth	16.5
Driller	EDI	Well diameter	N/A
Page 1 of 1 — Elevation	322	Well depth	N/A

LITHOLOGY / DESCRIPTION	U.S.C.	Sample (Recovery / Interval)	Static Water Level	Depth (feet)
Brownish-gray mottled silty sand to sandy silt (medium stiff to loose, moist)	SM/ML			3
Brownish-gray silty fine to medium sand with trace gravel (medium dense, moist) **(Glacial Drift)**	SM			5.5
Brownish-gray fine to medium sand with silt and trace gravel (very dense, moist) **(Glacial Drift)**	SP-SM			10.5
Brown fine to medium sand with silt (very dense, moist)	SP-SM			15.5

Boring was completed at 16.5 feet on 2/22/2013
Groundwater was not encountered

Standard Penetration Resistance
(140 lb. weight, 30" drop)
◆ Blows per foot
■ Moisture Content



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Woodinville, Washington 98072

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Figure 4

B-2	Date	2/22/2013	Hole diameter	
	Logged by	KHB	Hole depth	21.5'
	Driller	EDI	Well diameter	N/A
Page 1 of 1	Elevation	321	Well depth	N/A

LITHOLOGY / DESCRIPTION

Lithology / Description	U.S.C.
Dark brown sandy silt with organics and roots (loose, moist)	ML
Brownish-gray slightly mottled sandy silt with trace roots (hard, moist) **(Glacial Drift)**	ML
Brown silty fine sand with trace gravel (dense, moist) **(Glacial Drift)**	SM
Brown fine to medium sand with silt (dense, moist) **(Glacial Drift)**	SP-SM
Brown fine to medium sand with silt, trace silt clasts and trace wood (dense, wet) **(Glacial Drift)**	SP-SM

Boring was completed at 21.5 feet on 2/22/2013
Groundwater was encountered at 19.0 feet

Sample: Recovery, Interval

Static Water Level

Depth (feet): 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25

Standard Penetration Resistance
(140 lb. weight, 30" drop)
◆ Blows per foot
■ Moisture Content

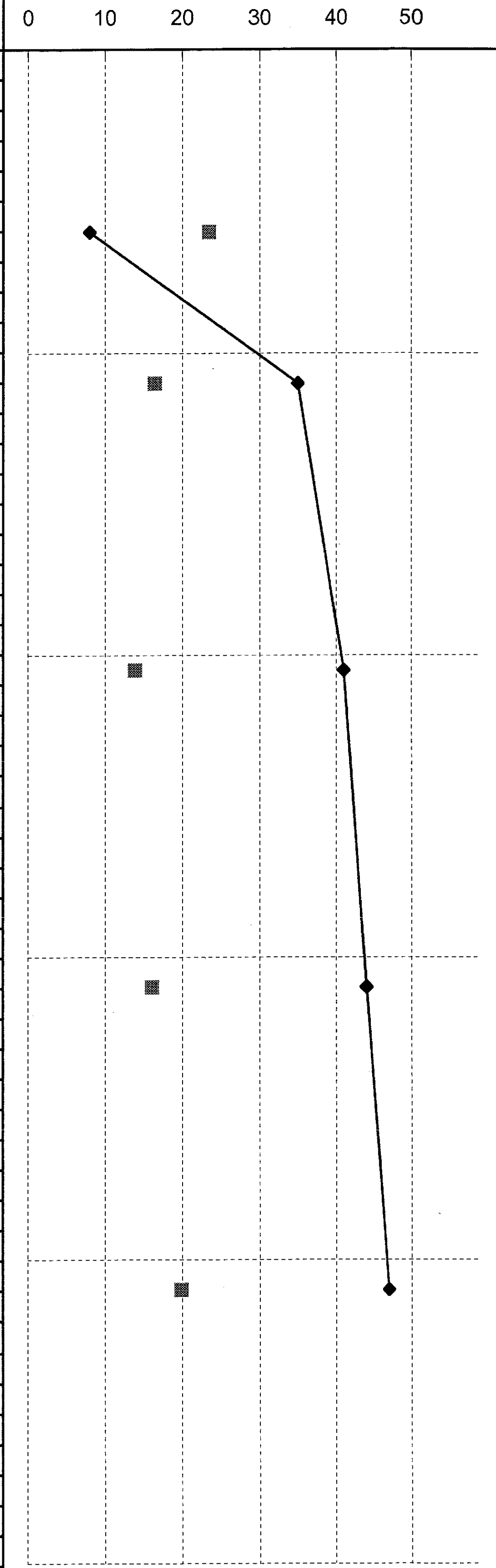

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Figure 5

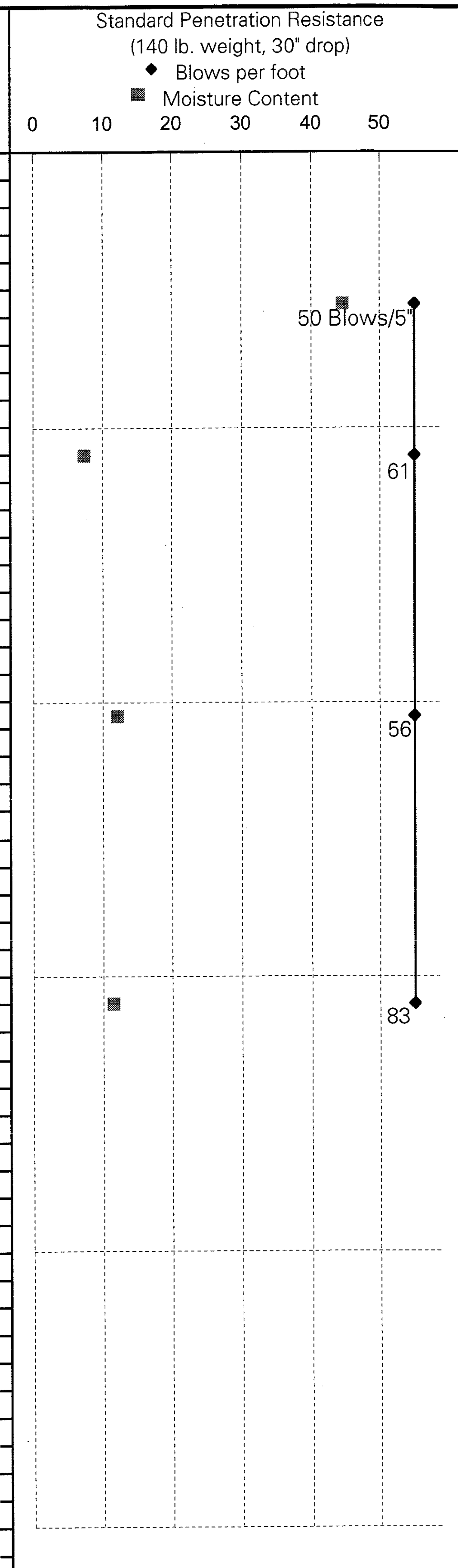
B-3

Page 1 of 1

Date	2/22/2013	Hole diameter	
Logged by	KHB	Hole depth	16.5'
Driller	EDI	Well diameter	N/A
Elevation	327	Well depth	N/A

LITHOLOGY / DESCRIPTION	U.S.C.	Sample (Recovery / Interval)	Static Water Level	Depth (feet)
Brown silty sand to sandy silt with roots and trace gravel (soft to loose, wet) (blow counts overstated due to gravel in shoe of sampler)	SM/ML			3
Brownish-gray fine to coarse silty sand with gravel (very dense, moist) **(Glacial Drift)**	SM			5.5
Brownish-gray fine to medium sand with silt (very dense, moist) **(Glacial Drift)**	SP-SM			10.5
Brownish-gray fine to medium sand with trace gravel (very dense, wet) **(Glacial Drift)**	SM		▽	15.5

Boring was completed at 16.5 feet on 2/22/2013
Groundwater was encountered at 15.0 feet



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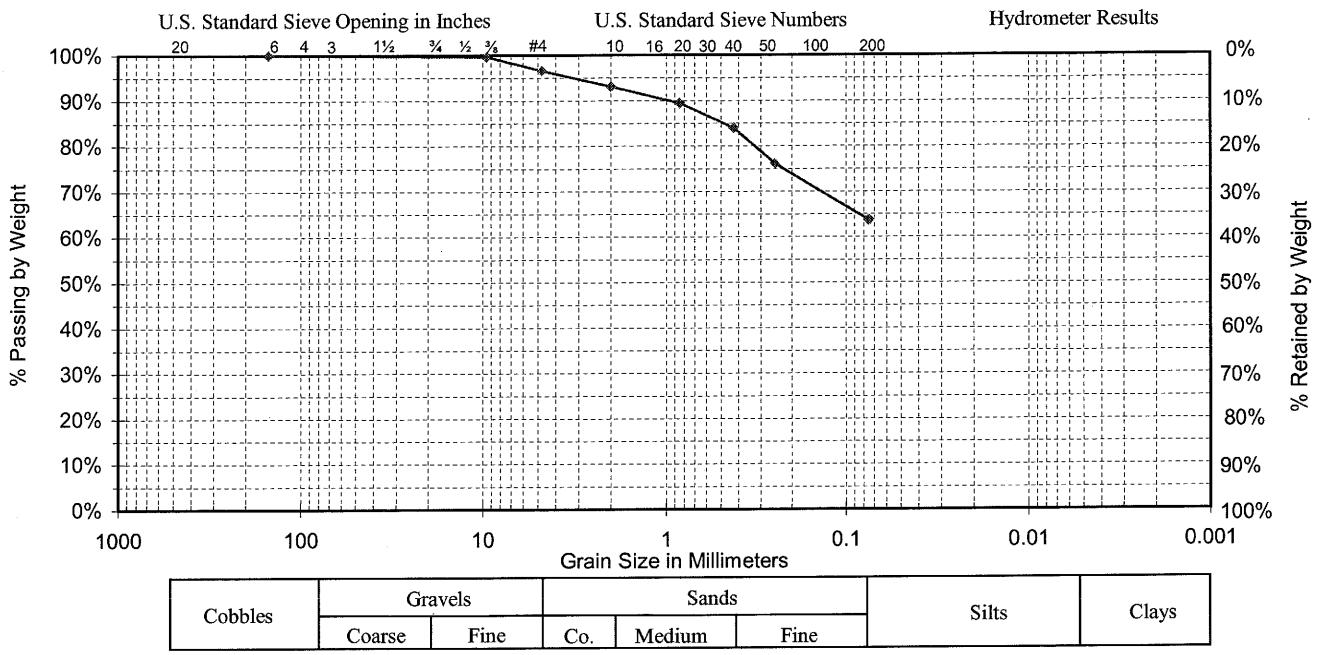
Anjuman-E-Burhani Community Complex

2791-001A

Figure 6



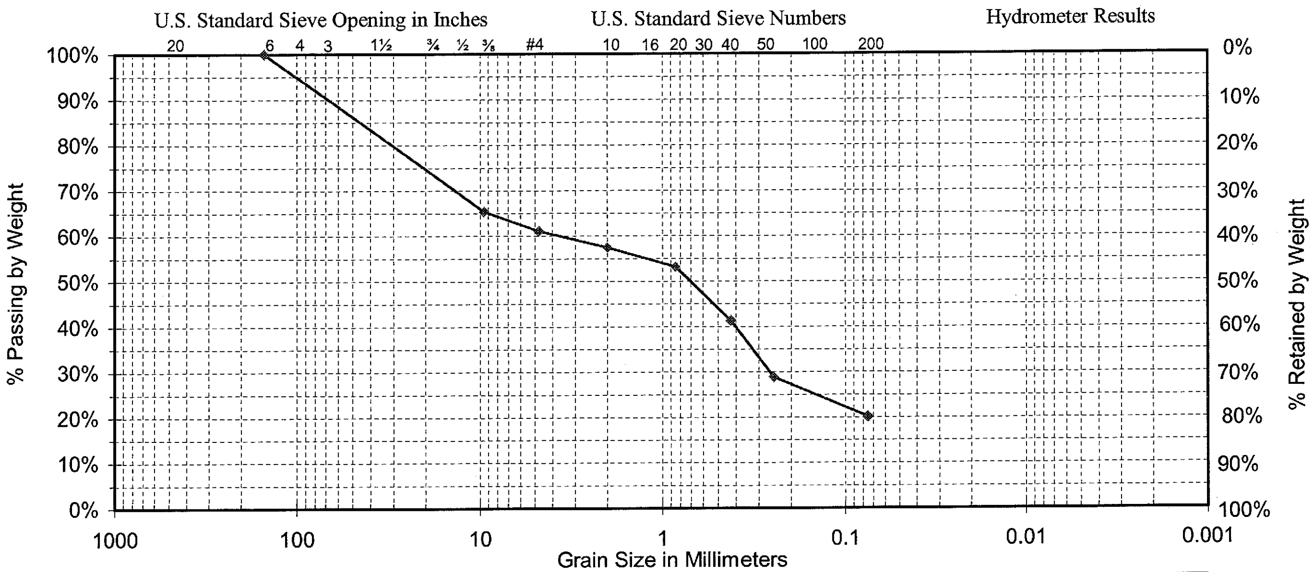
Date :	D_{10} = 0.01	**USCS Classification**	% Gravel 3.5% / % Sand 32.8%
Sample #: 2	D_{30} = 0.04	ML, Sandy Silt	
Sample ID:	D_{60} = 0.07	**Specifications**	
Source:	C_C = 1.50	No Specs	
Project: 2791-001A	C_U = 6.00	Sample Meets Specs	% Silt & Clay
Location: Redmond, WA	Liquid Limit= 0.0%	No	63.7%
Boring #: 2	Plastic Limit= 0.0%	Fineness Modulus	
Depth: 5 to 6.5	Plasticity Index= 0.0%	0.86	

Coarse Section

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
6.00"	150.00		100.0%		
4.00"	100.00		100.0%		
3.00"	75.00		100.0%		
2.50"	63.00		100.0%		
2.00"	50.00		100.0%		
1.75"	45.00		100.0%		
1.50"	37.50		100.0%		
1.25"	31.50		100.0%		
1.00"	25.00		100.0%		
7/8"	22.40		100.0%		
3/4"	19.00		100.0%		
5/8"	16.00		100.0%		
1/2"	12.50		100.0%		
3/8"	9.50	99.6%	99.6%		
1/4"	6.30		97.5%		
#4	4.75	96.5%	96.5%		

Fines Section

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
#4	4.750	96.5%	96.5%		
#8	2.360		93.5%		
#10	2.000	93.1%	93.1%		
#16	1.180		90.5%		
#20	0.850	89.4%	89.4%		
#30	0.600		86.2%		
#40	0.425	83.9%	83.9%		
#50	0.300		78.3%		
#60	0.250	76.1%	76.1%		
#80	0.180		71.1%		
#100	0.150		69.0%		
#140	0.106		65.9%		
#170	0.090		64.7%		
#200	0.075	63.7%	63.7%		
#270	0.053				



Cobbles	Gravels		Sands			Silts	Clays
	Coarse	Fine	Co.	Medium	Fine		

Date :		D_{10} = 0.04	**USCS Classification**	% Gravel	% Sand
Sample #:	2	D_{30} = 0.27	SM, Silty Sand with Grav	38.9%	41.0%
Sample ID:		D_{60} = 3.93	**Specifications**		
Source:		C_C = 0.48	No Specs		
Project:	2791-001A	C_U = 105.22	Sample Meets Specs	% Silt & Clay	
Location:	Redmond, WA	Liquid Limit= 0.0%	No	20.1%	
Boring #:	3	Plastic Limit= 0.0%	Fineness Modulus		
Depth:	5 to 6.5	Plasticity Index= 0.0%	3.59		

Coarse Section

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
6.00"	150.00		100.0%		
4.00"	100.00		100.0%		
3.00"	75.00		100.0%		
2.50"	63.00		100.0%		
2.00"	50.00		100.0%		
1.75"	45.00		100.0%		
1.50"	37.50		100.0%		
1.25"	31.50		100.0%		
1.00"	25.00		100.0%		
7/8"	22.40		100.0%		
3/4"	19.00		100.0%		
5/8"	16.00		100.0%		
1/2"	12.50		100.0%		
3/8"	9.50	65.3%	65.3%		
1/4"	6.30		62.5%		
#4	4.75	61.1%	61.1%		

Fines Section

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
#4	4.750	61.1%	61.1%		
#8	2.360		57.9%		
#10	2.000	57.4%	57.4%		
#16	1.180		54.4%		
#20	0.850	53.2%	53.2%		
#30	0.600		46.2%		
#40	0.425	41.3%	41.3%		
#50	0.300		32.4%		
#60	0.250	28.8%	28.8%		
#80	0.180		25.3%		
#100	0.150		23.8%		
#140	0.106		21.7%		
#170	0.090		20.9%		
#200	0.075	20.1%	20.1%		
#270	0.053				

Appendix A

USGS Design Maps Summary Report

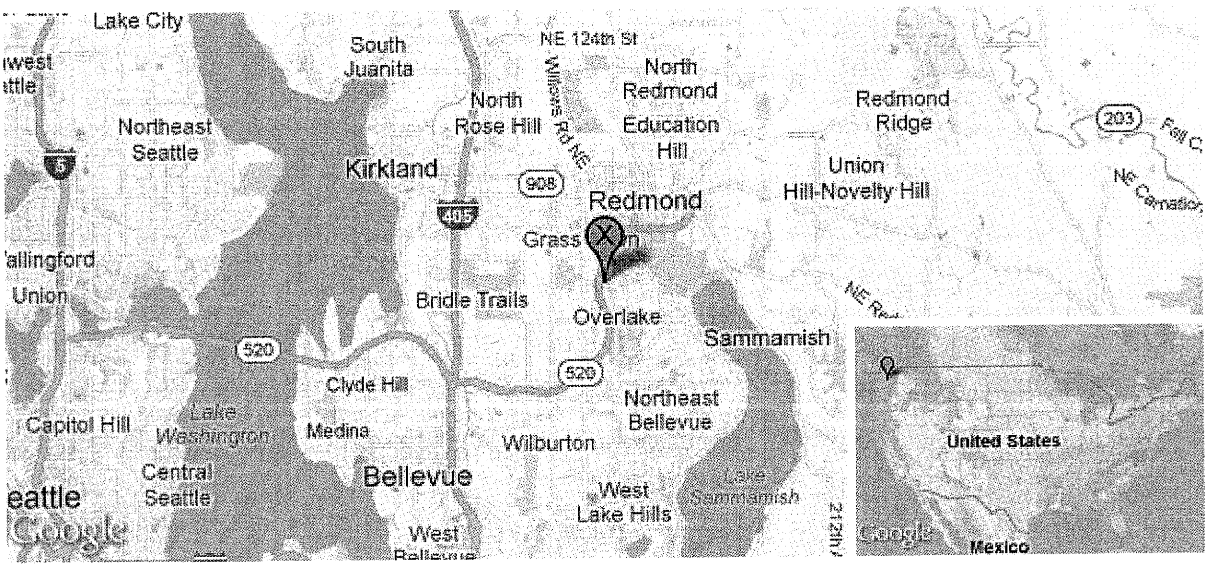
User–Specified Input

Building Code Reference Document 2012 International Building Code
(which makes use of 2008 USGS hazard data)

Site Coordinates 47.65477°N, 122.13595°W

Site Soil Classification Site Class C – "Very Dense Soil and Soft Rock"

Risk Category I/II/III

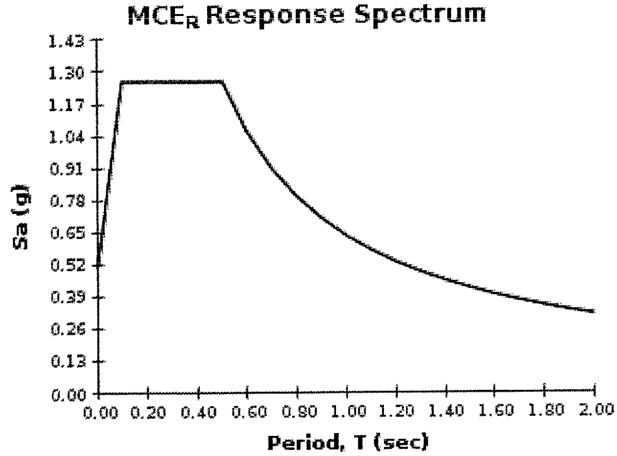


USGS–Provided Output

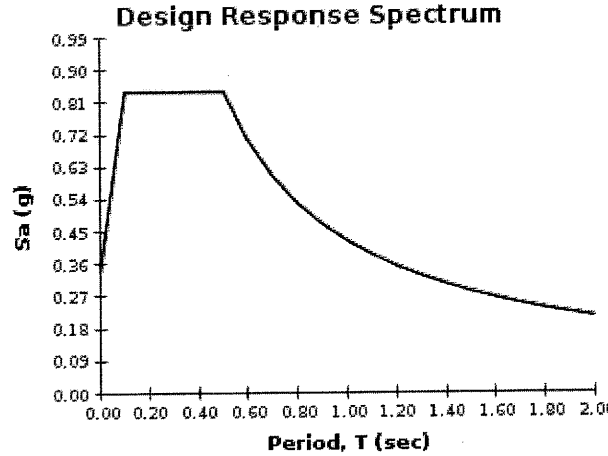
$S_S =$	1.259 g	$S_{MS} =$	1.259 g	$S_{DS} =$	0.839 g
$S_1 =$	0.482 g	$S_{M1} =$	0.635 g	$S_{D1} =$	0.424 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



USGS Design Maps Summary Report

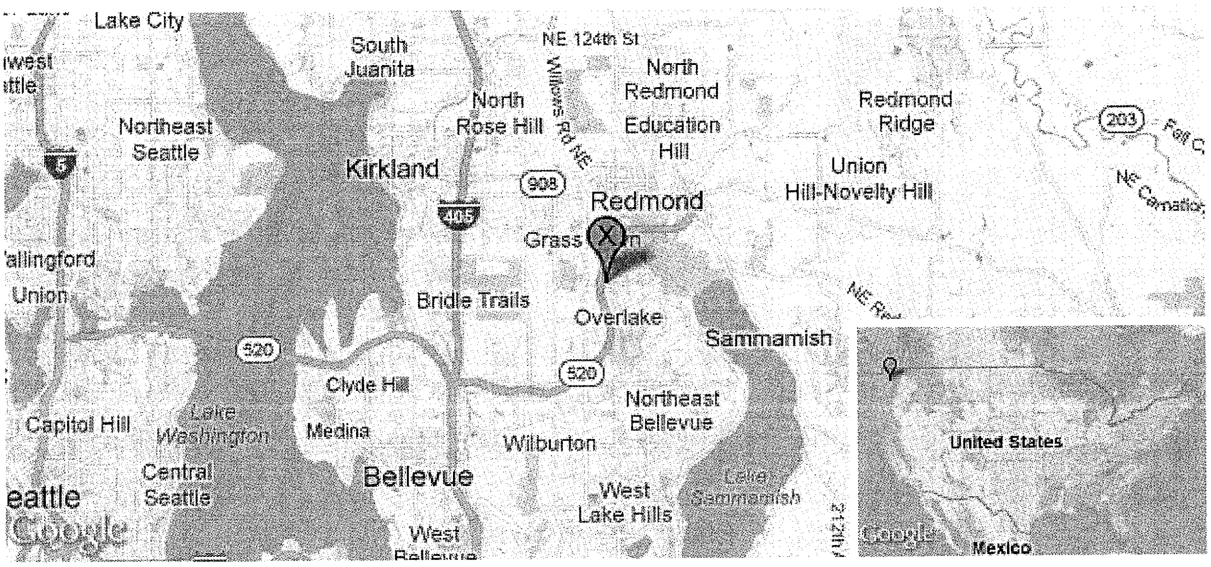
User–Specified Input

Building Code Reference Document 2006/2009 International Building Code
(which makes use of 2002 USGS hazard data)

Site Coordinates 47.65477°N, 122.13595°W

Site Soil Classification Site Class C – "Very Dense Soil and Soft Rock"

Occupancy Category Occupancy Category I



USGS–Provided Output

$S_S =$	1.249 g	$S_{MS} =$	1.249 g	$S_{DS} =$	0.833 g
$S_1 =$	0.422 g	$S_{M1} =$	0.581 g	$S_{D1} =$	0.387 g